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DISCUSSIONS

APPLICATIONS FOR ADMISSION AND TRANSFER

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MEMBERSHIP

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852
PAPERS

HIGH DAMS ON PERVIOUS GLACIAL DRIFT

By EDWARD M. BURD[1], M. AM. SOC. C. E.

SYNOPSIS

Dealing in general with high dams on pervious glacial drift, the subject of this paper is based, to large extent, on Hardy Dam, on the Muskegon River, in Michigan. It is also based on the generalities of design and construction developed for such foundation by thirty years' experience with many similar structures of constantly increasing head and magnitude. This dam, which creates a 100-ft head, was placed in operation at full head on April 30, 1931. It is composed essentially of a porous sand embankment, 120 ft high, with a central flexible concrete core-wall. The core-wall is set on steel sheet-piling driven 60 ft into underlying, pervious, and heavily water-bearing sand and gravel glacial drift several hundred feet thick over bed-rock. Three penstocks, 14 ft in diameter, which are used both for power and spilled water, extend through the base of the deep section.

This type of construction, built on this kind of foundation material, is unusual. The difficulties involved have been solved mostly by experiment. The paper concerns matters of general design interest under similar conditions, such as foundation and embankment settlement, flexibility of structures, percolation, and spillway construction. Furthermore (and of equal or even greater importance), improvements and economics of design are suggested as necessary prerequisites to continued power development under the existing natural conditions and economic limitations.

INTRODUCTION

Dams should preferably be built on rock foundations—good rock foundations—as emphasized by past and present experience. Circumstances and conditions, however, have compelled the construction of many notable structures on various kinds of earth, and, notwithstanding the Biblical injunction,

NOTE.—Discussion on this paper will be closed in August, 1933, *Proceedings*.

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the horrible example therein described has been avoided in a commendably large percentage of such situations. Engineering in an area of glacial dumping grounds is largely a contest with this problem.

While many regions of the earth are devoid of accessible rock foundation, there is a belt across the eastern part of the border between the United States and Canada that has peculiarities of its own. Glacial drift is extremely deep, being as much as 1 500 ft in Michigan. While its deposition followed natural laws, the results are heterogeneous and seemingly at random to such an extent that any except the most general deductions beyond actual exploration are wholly unreliable. Most of the deposit was water-washed so thoroughly that the fines were removed and the residue became coarse and permeable. In this region natural drainage is geologically new and undeveloped, and groundwater lies close to the surface, usually permeates the entire deposit, and flows through it. Precipitation is fairly high, streams are swift, flow is unusually uniform, and water power markets are close at hand, all of which tends to foster dam construction on this unfavorable geologic structure.

Log-driving dams first, and, later, innumerable mill dams, all of moderate head, have been common throughout this region for fifty years. Many still survive, large numbers have been washed out for one reason or another, some were abandoned, and very few are still of economic benefit. None of them ever developed into structural problems in the field herein discussed, although a fair proportion have been re-developed into electric power properties on a moderate scale.

At the turn of the century, old mill dams on the Kalamazoo River, in Michigan, had been rebuilt into generating stations of the type in which a series of vertical-shaft turbines in an open timber penstock drove a single horizontal shaft with a generator at one end. Each turbine was harnessed through its large wooden-toothed bevel gear to a driven iron pinion on the main shaft. Over-all efficiency was not more than 50%; noise and vibration was considerably more than 200%; and the alignment of a long horizontal shaft over eight to twelve turbines, even if it was set straight (with penstock empty), on its log crib and earth foundations, became something fearful and wonderful with the penstock filled. Increasing power requirements of the industrial development of the Lower Peninsula of Michigan induced the design and construction of constantly increasing heads on the Kalamazoo, Grand, Muskegon, Au Sable, and Manistee Rivers, in the order named. In this evolution a center core-wall type of sand embankment on sand foundation was developed under the leadership of William G. Fargo, M. Am. Soc. C. E., for heads up to 48 ft, which was the limit beyond which dam construction, under existing conditions, was then considered impractical. Since then the Company with which the writer is connected, has finished a number of additional projects, modifying the design considerably as study and experience indicated. A plant of 41-ft head has been built on quicksand, another of 71-ft standing head was completed on clay and sand on the Manistee River, in 1926, and the plant on the Muskegon River, at Hardy, Mich. (100-ft head) was put into operation early in 1931. The latter design has a set of three pressure penstocks, 14 ft in diameter, on the original river bed through the deepest

part of the embankment, and the foundation is of sand and gravel typical of an outwash plain. It is heavily permeated with water and its depth (perhaps 500 ft to bed-rock) has not been determined.

Topographically, still higher heads are possible on sites yet undeveloped. Structurally, present practise must apparently be considerably modified to adapt the design to higher heads, which, however, appear feasible and which offer a most interesting problem. Economically, the advance of steam generation has reduced the hydro-electric field to that of supplying peak power. This requires large ponds, high heads, large machinery capacity, and points toward future use of back-pumping during slack periods to supply water power for the next peak. Of course, water-power plants were built on the cheapest sites first, leaving each successive site less attractive as an investment unless new and more economical ideas could be devised. This general problem of geology, topography, design, and finance faces not only Michigan, but Wisconsin, Northern New York, and parts of Maine and Canada. Undeveloped water power approximately equalling those developed awaits its feasible solution. The purpose of this paper is to give in essential detail the experience developed in Michigan to date, and to suggest perhaps some possible changes and improvements.

Settlement of Supporting Earth Foundations

The formation of river beds in Michigan usually consists of a winding channel eroded from a glacial outwash plain, frequently following, in a general way, the toe of a terminal moraine which probably established the drainage arrangement originally. These outwash plains consist mostly of stratified, water-washed sands and gravels in all degrees of fineness, normally alternating with layers of so-called "mudstone." This latter material consists of water-borne glacial silt and rock flour deposited during periods of glacial back-waters caused by changes that are now more or less obscure. The weight of over-burden and the passage of time have compressed the mudstone layers into dense, tough, impervious beds into which a pick point can be driven only a few inches. When excavated, even in stream beds, the material is merely damp, not wet. When placed in water it disintegrates very slowly, but when pulverized and made into mud its rate of settlement and consolidation is a matter of geological time; for practical purposes "mudstone" remains fluid indefinitely. A 40-lb water-jet playing squarely against an exposed but undisturbed face for a week cuts away only a few inches. This material is distinctly different from morainal till clay deposits or clay resulting from rock distintegration in place. Boulders are almost never found in it, although occasionally they are found in the sand and gravel layers, where apparently they have been rolled along a stream bed. Casual observation may often show the present stream bed to be covered with a great accumulation of stones from material that has been washed away.

The foregoing describes the sections of the larger rivers in the Lower Peninsula of Michigan that are suitable for power development. Their smaller branches and other smaller streams flow through and over all types of glacial deposits, which fill the rock-stratum basin of the Lower Peninsula

to a maximum depth of 1 500 ft. Lakes Michigan and Huron are merely glacial gouges from this deposit. Even the river beds north of the Grand Rapids-Saginaw line are elevated many hundred feet above ledge by a heterogeneous glacial dump that has no predictable formation or uniformity from point to point. Frequently, it is coarse and porous and heavily permeated with water. Nearer the surface the action that formed the outwash plain has given some continuity to the various strata, but, nevertheless, the mudstone layers are often irregular and fragmentary, due to subsequent forward glacial shoves or stream erosion after their initial deposition.

The selection of dam sites on this formation is largely a matter of arranging them so as to fit available mudstone, in the best possible manner, for foundation purposes. Surface topography in a river valley may be unrelated to the mudstone strata. In many cases the rivers have cut through these strata where they are most needed, and a series of ponds may necessitate a plant where only a sand foundation is available. Foundation design and safe head are largely matters of experience and necessity, rather than theory. Safety requires building for the worst existing condition as determined by borings, surface study, and previous experience.

Settlement under load on this formation fortunately proves less serious than might be expected, probably due mostly to previous consolidation under greater load before the stream eroded the valley, and partly to the water-settled nature of the material in outwash plains. The latter is an important factor. Material in the outwash plains is far different in behavior from that of till clays and unwashed morainal deposits, which have a much greater and uncertain range of settlement. Although test piles and test loading are used in each design, for well known reasons their indications are frequently inconsistent with actual construction adjustment, which constitutes the data herein presented.

Foundation settlement records on a number of dams in the region during the twenty years (1913 to 1932, inclusive) are typical of data shown in Figs. 1, 2, and 3, and in Table 1. These three illustrations and the table constitute the record to the end of 1932, of a 100-ft head partly on a thin stratum of "mudstone" supported on pervious water-bearing sand and gravel, probably 400 or 500 ft thick to bed-rock. The loading shown on Fig. 1 is the most severe yet imposed on this formation. Every detail was studied to avoid a large difference in load intensity at any point. Uniform variation of uplift as indicated on Fig. 1 has been determined experimentally by open-well pipes across the section of several lower but similar embankments. Borings were taken by "dry-sampling" pipes driven beyond the bottom end of 3-in. casings which were sunk by well-driving methods. The most pronounced difference is at the down-stream edge of the power-house block, where the expected differential settlement of about $\frac{3}{8}$ in. occurred. The maximum of about 3 in. under the core-wall was only one-half that expected, as explained subsequently under the heading "Design of Foundation Structures."

Settlements in Table 1 are total cumulative values, in feet, beginning at the date marked by an asterisk (*), for the years 1930, 1931, and 1932. Begin-

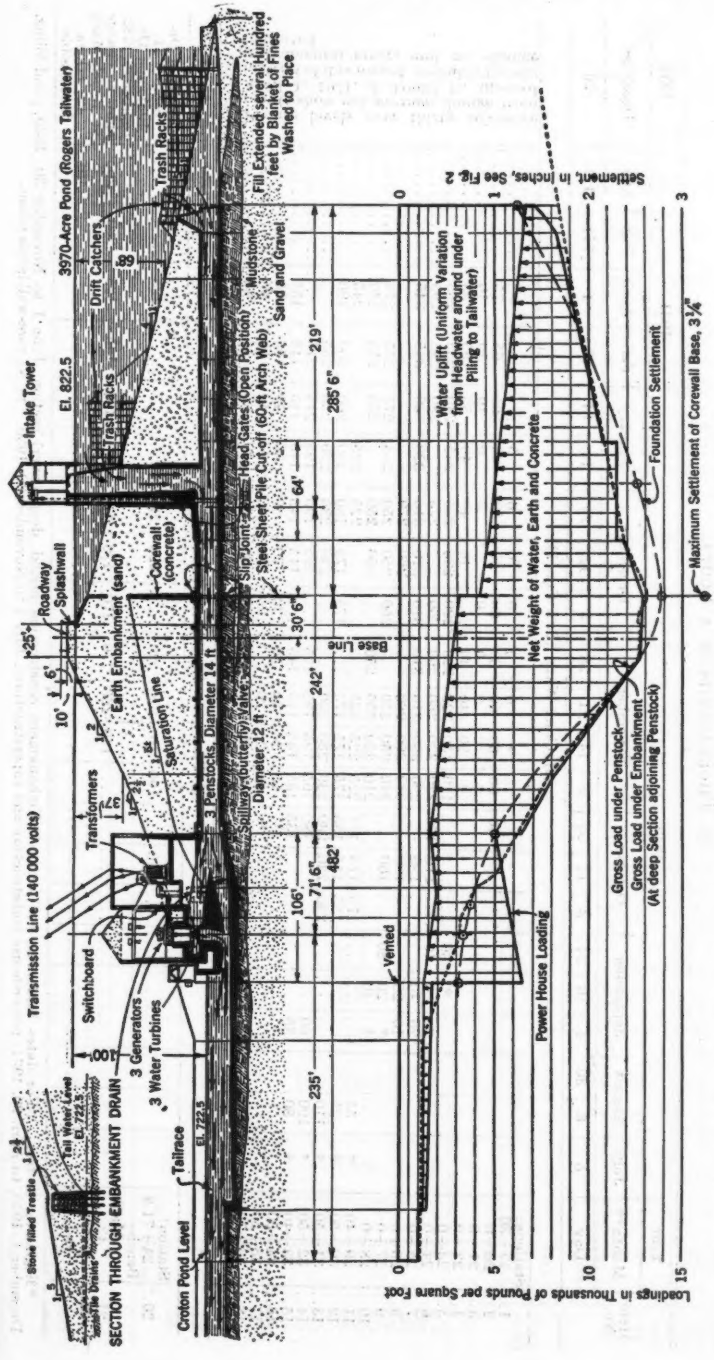

FIG. 11.—CROSS-SECTION AND BASE LOADING OF HARDY DAM

TABLE 1.—SETTLEMENT RECORD; FOUNDATION, HARDY DAM, MICHIGAN (CUMULATIVE SETTLEMENT OBSERVATIONS, IN THOUSANDTHS OF A FOOT)

Item No.	Year	1930																1931						1932
	Month**	July	August		September			October			November				December			January		Feb.	April	July	Aug.	December
	Day	5	6	30	4	10	24	3	17	30	7	14	21	28	5	12	29	13	31	27	1	7††	1††	29
	(a) CORE-WALL (SEE FIG. 2)																							
	Stations:																							
1..	19+30											*	0	0	0	0	0	5	....	....	....	....	....	Precise levels over thirty reference points show net average change from April 1, 1931, of 0.0035 ft, upward instead of downward, probably merely instrumental errors and no change indicated
2..	20+0							*	....	60	60	60	60	60	60	60	60	60	65	65	75	....	....	
3..	21+0										*	0	0	0	0	....	0	0	10	10	10	....	....	
4..	22+0										*	20	40	40	....	40	40	40	50	55	55	....	....	
5..	23+0					*	30	....	....	....	110	150	220	220	220	230	235	235	240	255	255	....	....	
6..	24+0				*	0	10	....	110	170	220	220	260	260	260	....	275	....	280	295	295	....	....	
7..	25+0			*	20	40	....	....	190	....	290	300	310	310	310	310	310	320	330	345	345	....	....	
8..	26+0			*	10	20	....	....	190	....	240	260	260	....	....	260	270	....	....	....	....	....	....	
9..	27+0				*	20	80	....	160	....	240	250	250	....	250	250	250	255	275	275	275	....	....	
10..	28+0				*	40	....	....	....	....	210	250	250	250	....	250	250	....	250	250	250	....	....	
11..	29+40	*	55	....	....	....	....	175	175	175	225	225	225	....	....	....	225	225	....	....	245	....	....	
12..	30+50	*	35	....	....	....	....	155	155	155	175	175	175	....	175	175	180	180	200	200	190	....	....	
13..	31+50	*	110	....	....	....	....	230	230	230	230	230	230	....	....	230	230	230	255	255	....	....	....	
14..	32+50	*	100	....	160	....	....	....	....	180	195	195	195	195	....	195	195	195	215	215	....	....	....	
15..	33+40	*	80	....	135	....	....	....	....	135	135	135	145	145	....	145	145	145	145	145	155	....	....	
16..	34+50									*	10	20	30	30	30	30	30	30	40	55	35	....	....	
17..	35+50									*	10	20	20	....	20	20	20	20	40	50	45	....	....	
18..	36+70									*	20	20	20	20	20	20	....	20	45	45	45	....	....	
19..	38+10									*	35	35	45	45	45	50	....	50	60	60	....	....	....	
	(b) PENSTOCK (SEE FIGS. 1 AND 2)																							
20..	Station: 24+71.8				*	....	....	....	110	....	190	210	220	220	....	220	220	....	....	....	225	....	....	Levels of December 31, 1931, and December 29, 1932, show no change at Bench-Mark 42; others inaccessible
21..	Bench-Mark: 42	‡	....	....	....	....	....	....	....	....	....	140	160	170	170	170	170	195	195	195	195	200	205	
22..	Up-stream end	‡	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	105	

* Beginning of test. ‡ For dates beginning, see text. ** Embankment construction period, deep (river) section, June 1 to November 30, 1930; pond filling, December 1, 1930, to April 30, 1931; power-house (substructure and superstructure), July 1 to November 30, 1930. †† No core-wall levels taken.

TABLE 1.—*Continued*

TABLE 1.—*Continued*

Item No.	Year	1930														1931									1932
	Month**	September		October					November				December			January		Feb.	April	May	June	July	August	Dec.	December
	Day	20	25	4	14	23	27	31	7	14	21	28	5	13	31	15	30	26	1	1	1	7	1	21	29
	(c) POWER HOUSE (SEE FIG. 3)																								
	Bench-Marks:																								
23..	11								*	5	5	15	20	20	20	20	20	25	25	10	25	30	35	32	Bench-marks 11, 12, 13, 4, 5, 6, 7, 10, and two turning points average 0.003 ft higher; probably instrumental error; no change indicated.
24..	12								*	0	0	15	20	20	20	20	20	30	25	25	30	30	25	35	
25..	13								*	0	5	15	15	15	15	15	15	30	25	25	20	25	25	29	
26..	14	‡	....	....	....	....	....	50	60	60	60	65	‖												
27..	15	‡	....	....	....	....	....	60	70	70	70	75	‖												
28..	16	‡	....	....	....	....	....	50	70	70	70	70	‖												
29..	4								*	5	5	....	5	5	10	10	10	15	20	20	20	20	20	22	
30..	5								*	4	5	1	15	2	20	20	20	25	....	20	15	15	15	13	
31..	6								*	0	5	5	10	15	15	15	15	15	5	15	5	15	10	16	
32..	8								*	0	5	−5†	20	20	20	20	20	25	15	15	15	20	10	....	
33..	17	‡	30	....	....	....	....	60	55	55	55	§													
34..	1	‡	....	0	....	20	20	....	40	40	40	40	50	50	§										
35..	2	‡	....	30	30	30	....	....	40	40	40	50	....	50	....	....	60	....	55	§					
36..	3	*	....	15	....	30	....	....	45	....	....	....	....	....	55	§									
37..	7								*	0	0	0	5	5	10	10	10	10	15	15	15	10	10	12	
38..	9								*	0	0	5	....	20	20	20	....	....	0	0	....	....	....	....	
39..	10							....	*	5	5	5	5	0	−5†	−5†	5	10	....	10	15	5	5	14	
40..	18	‡	....	....	....	....	....	10	30	30	40	45	§												

(d) EMBANKMENT (SEE FIG. 2)

Item No.	Year	1930								1931				
	Month**	November					December			January		Feb-	April	May
	Day	2	7	14	21	28	5	15	30	15	31	27	1	1
	Bench-Marks:													
41..	2	*	0	0	15	15	15	20	45	45	45	50¶		
42..	3						*	0	0	0	5	25¶		
43..	4						*	0	0	0	0	25¶		
44..	5						*	0	30	40	40	60¶		
45..	....											....		
46..	16	*	0	0	30	55	85	110	140	150	160	160	150	195
47..	17	*	15	40	40	40	45	45	55	55	90	90	105	115
48..	19	*	0	20	20	60	75	75	75	85	150	150	155	165
49..	21						*	5	50	90	100	135	....	155

Item No.	Year	1930			1931					1932
	Month**	December			January		Feb.	April	May	Dec.
	Day	5	15	30	15	31	27	1	1	29
	Bench-Marks:									
50...	23	*	10	35	40	60	60	95	95	Same as for "Core-Wall," no change. (For all Items Nos. 41–57)
51...	25	*	45	80	105	145	145	155	160	
52...	27	*	30	30	35	45	60	40	60	
53...	34	*	5	25	25	25	25	40	55	
54...	36	*	0	20	20	20	20	30	45	
55...	38	*	0	15	15	25	25	40	65	
56...	39	*	10	25	25	35	35	45	60	
57...	40	*	15	30	30	35	35	55	65	

* Beginning of test. † Bench-mark reported higher than it was initially (all readings in general being downward). ‡ For dates beginning, see text. § Covered by construction. ‖ Covered by fill; record transferred to Items Nos. 23, 24 and 25. ¶ Flooded out. ** Embankment construction period, deep (river) section, June 1 to November 30, 1930; pond filling, December 1, 1930, to April 30, 1931; power-house (substructure and superstructure), July 1 to November 30, 1930.

ning dates are not shown for nine items in the table because of lack of room. To complete the record, these are: Items Nos. 21 and 22, March 25, 1930; Items Nos. 26, 27, 28, and 40, August 15, 1930; Item No. 34, August 30, 1930; Item No. 35, September 2, 1930; and Item No. 33, September 17, 1930.

It will be noted that, by averaging the final observations for Items Nos. 5 to 11, the average total settlement of the foundation was 0.28 ft. Levels taken at the penstock (Table 1(*b*)), on December 30, 1931, showed that there had been no change since August 1 of that year. Total settlements on the power house, as computed from Table 1(*c*), were as follows: Up stream, 0.085-ft; center line of power-house block, 0.065 ft; center line of units, 0.055 ft; and down stream, 0.050 ft. Likewise, the embankment (Table 1(*d*)) settled as follows: Items Nos. 46 to 49 (average), 0.160 ft; Items Nos. 50 to 52, 0.105 ft; and Items Nos. 53 to 57, 0.060 ft.

The foundations of the intake, penstocks, and power house were arranged on the only available mudstone, a bed about 20 ft thick, which "pinched out" both up stream and in the river channel, being the remains of a 40-ft stratum through which the river had cut. Such an arrangement is largely a matter of construction convenience. As far as load-bearing is concerned it serves merely as a pad or cushion upon the underlying sand, to smooth out local variations. Its effect in this regard is represented by about $\frac{1}{2}$-in. difference between $3\frac{5}{16}$-in. settlement of the core-wall foundation on sand and gravel and $2\frac{3}{4}$-in. settlement of the penstock foundation on this mudstone for practically the same loading. Total settlement to date (1933) is more nearly in proportion to gross loading intensity than net, which may indicate that water uplift is less than the uniformly varying pressure shown. This is probably true for the foundation on mudstone, but it is a condition unlikely to occur in porous sand.

Details of these settlement records are shown on Fig. 2, and in Table 1, which give the pertinent and typical points selected from a large number of field data. The trend of this record shows that settlement takes place in proportion to load. The embankment was all "roughed in" by November 30, 1930. After that date, and while the pond was filling, additional settlement of the core-wall base was only about $\frac{1}{4}$ in. More movement than this was expected while the pond was filling. Apparently, the uplift diminished the added water load considerably. The power house was completed by the end of 1930, as far as loading was concerned, the machinery and superstructure being finished early in 1931. It did not settle in direct proportion to its load, apparently being affected and controlled by general embankment settlement around it. (See Fig. 3 and Table 1(*c*).) No doubt the settlement of the embankment threw a heavy additional load on the up-stream side and may have reversed its base loading, which was desirable, but not assured by the design. At of the end of 1932 there has been no further foundation settlement. In general, the records show that settlement closely follows the loading and unlike more normal soils, including clays, settlement is not only less, but ceases much earlier. Fig. 4 shows these generalities which are typical of this kind of construction.

The field method of obtaining these data was to establish reference points on the concrete base as soon as each "pour" was completed, tying them in carefully with several scattered permanent bench-marks well away from all con-

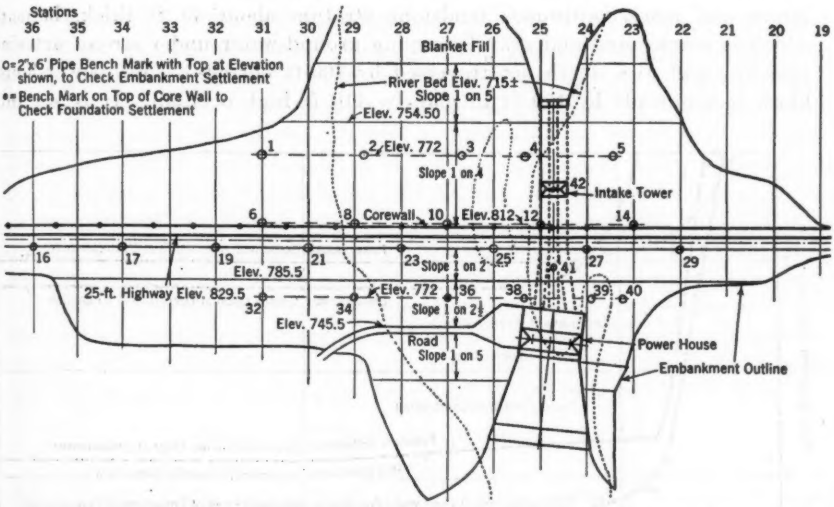

FIG. 2.—SETTLEMENT RECORD OF HARDY DAM; PLAN OF STATIONS AND BENCH-MARKS (SEE TABLE 1)

struction. Reference points were replaced on each successive pour, being carried in this way from the base to the crest of the core-wall, and assuming no appreciable change in the concrete itself after it was well set. The final points will continue to be observed until equilibrium has been established for a considerable time.

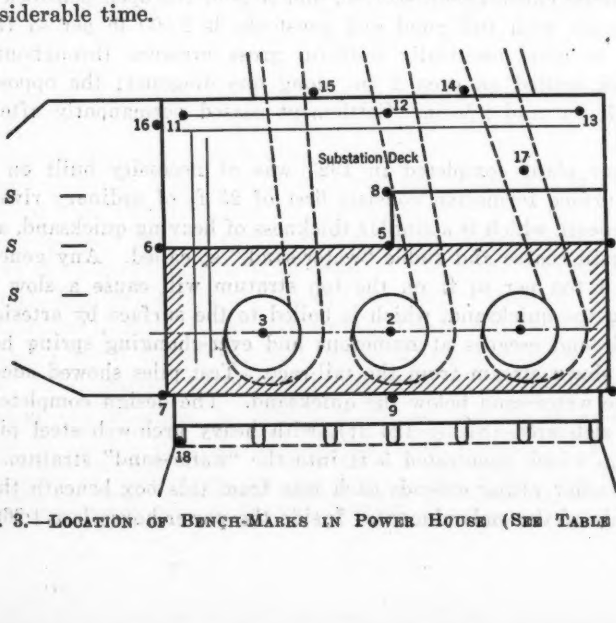

FIG. 3.—LOCATION OF BENCH-MARKS IN POWER HOUSE (SEE TABLE 1(c))

That settlement is proportional to load, and proceeds as loading is applied, and then ceases, even under these conditions, has been proved at several plants of lesser head, and under dissimilar conditions. In 1926, a plant of 71-ft head was completed on the Manistee River. It was founded upon a larger and more continuous mudstone stratum about 40 ft thick, beneath which is coarse sand and gravel carrying ground-water under strong artesian pressure, and to a depth not traversed by 100-ft borings. This power-house block is about 105 by 150 ft in plan by 110 ft high over all. It is set into

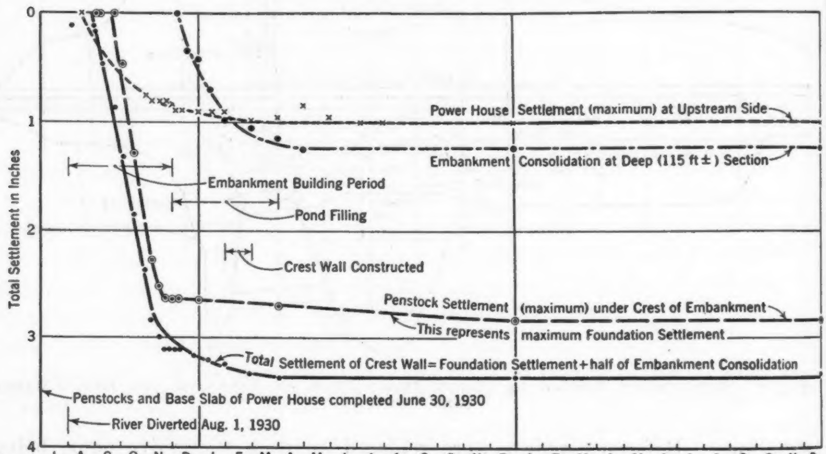

FIG. 4.—SETTLEMENT AT HARDY DAM

the maximum embankment section, and it is of the open penstock type. The base pressure with full pond and penstocks is 9 500 lb per sq ft, and it is designed to give practically uniform gross pressure throughout the base. This block settled an even 2 in. along one diagonal; the opposite corners settled $1\frac{15}{16}$ in. and $2\frac{1}{16}$ in. Settlement ceased permanently after the pond was filled.

Another plant completed in 1923 was of necessity built on quicksand. The subsurface formation consists first of 25 ft of ordinary river sand and gravel, beneath which is a similar thickness of heaving quicksand, and beneath this is again coarse and stable "water-sand," so-called. Any general loading of about 1 ton per sq ft on the top stratum will cause a slow settlement, displacing the quicksand, which is boiled to the surface by artesian pressure beneath it and escapes at numerous and ever-changing spring holes in the river bed down stream from the tail-race. Test piles showed adequate bearing in the water-sand below the quicksand. The design completely enclosed the base slab area (87 by 123 ft), with heavy arch-web steel piling, 55 to 60 ft long, which penetrated 5 ft into the "water-sand" stratum. A cut-off wall of similar piling extends each way from this box beneath the core-wall to the ends of the embankment. Inside the power-house box 1 266 long-leaf,

yellow pine piles, 70 ft long, were jetted down and then seated about 15 ft into the lower "water-sand" stratum to a 20-ton bearing, by the *Engineering News* formula, $R = \frac{2\,wh}{s + 0.1}$, using a double-acting steam hammer, in which, $wh = 4\,150$ ft-lb. These piles are spaced on 2 ft 8-in. centers, with minor variations, to take the total base-slab load which is 8 500 lb per sq ft up stream, and 8 000 lb per sq ft down stream. The top of the steel piling is cast into the base-slab concrete, 2 ft in from the edge. This construction, although quite different from that on good sand or mudstone, as before described, settled almost exactly 2 in. while the load was being applied; and no settlement occurred afterward. These and similar records have established a rule-of-thumb ratio of 1-in. settlement for each 5 000 lb per sq ft of gross loading for any good sand, gravel, or mudstone. How far this rule could be extended and whether sand-box loadings for higher intensities would give comparable results are as yet uncertain. Flexibility of design details at Hardy Dam were arranged for many times the actual settlement experienced. As far as load bearing and consequent settlement are concerned, it seems safe and practical to increase the height of future structures still more.

Design and Settlement of Embankments

To build embankments on deep glacial drift is usually a larger and more difficult part of the entire development than is generally true under more usual conditions. The foundation not only adjusts itself to load, but is highly pervious both to pond seepage and to natural ground-water from adjacent hills. The latter is frequently the more difficult to handle, particularly where the embankment joins the natural banks. Valleys are wide even at dam sites, especially for the higher heads. Spillway requirements are a minimum, as floods in the common meaning of the word are unknown. This combination of circumstances results in an embankment problem of relatively large magnitude, involving structural difficulties and costing from 25 to 40% of the entire outlay.

Embankment materials are restricted economically to sands and gravels typical of outwash plains, which are unlimited in quantity, and of good quality, except that they are coarse and pervious. This class of earth is always near at hand; easy to dig, transport, and place; but it is, of course, water-washed and, therefore, low in fines commonly used for cores. Fig. 5 shows the composite fineness curve of all borrow-pits for Hardy Dam, as well as analyses of such well-known earth dams as Alexander (Hawaii), Calaveras (California), Taylorville (Ohio), Miami Conservancy District (Ohio), Saluda (South Carolina), Blue Ridge (Georgia), Davis Bridge (Massachusetts), Bridgewater (North Carolina), Soft Maple (New York), and Don Martin (Mexico).[2] The curves permit a comparison of available embankment

[2] Hydro-Electric Handbook, by William P. Creager and Joel D. Justin, Members, Am. Soc. C. E., 1927, p. 249, Fig. 155; "Core Materials at the Germantown Dam," by C. H. Eiffert, M. Am. Soc. C. E., *Engineering News-Record*, December 18, 1930, pp. 954-958; "Materials in Existing Earth Dams," by E. W. Lane, M. Am. Soc. C. E., *Engineering News-Record*, December 18, 1930, pp. 961-965, and June 11, 1931, p. 962; and "Further Study of Earth Dams and Better Terminology Needed," by J. Albert Holmes, M. Am. Soc. C. E., *Engineering News-Record*, June 11, 1931, pp. 960-962.

material in outwash plains of the glaciated region with samples from borrow-pits and cores of important and representative dams, built by semi-hydraulic methods. The embankment of Soft Maple Dam, which is about as high as Hardy Dam and which is also in the glaciated area, shows the quality of hydraulic sluiced core that is obtainable from available glacial deposits. When samples were gathered and combined at Hardy Dam, a small quantity of mudstone was included to represent that unavoidably excavated at the pits

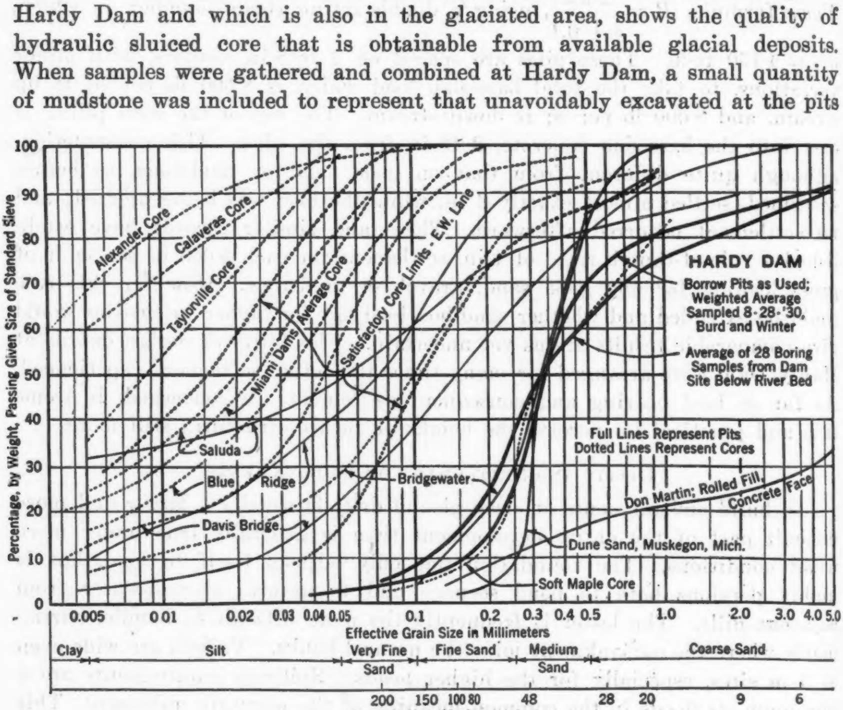

FIG. 5.—HARDY EMBANKMENT MATERIAL ANALYSIS

and transported to the dam, although the train haul and sluicing process could not pulverize these lumps and make them effective. Hence, the curve for Hardy Dam in Fig. 5 indicates more fines than actually are effective, and the curve for the dune sand at Muskegon, Mich., is perhaps more nearly typical. Even if all the fines could be washed out of the excavated material (which is impractical and uneconomical) the quantity would be inadequate for cores of the thickness ordinarily built by hydraulic or semi-hydraulic processes.

Only two embankments comparable to Hardy Dam have core or pit materials approaching its coarseness. Soft Maple Dam in Northern New York was built entirely by hydraulic-fill methods, but the similar available glacial material permitted a core scarcely any finer than the average of all the samples from the borrow-pits of Hardy Dam, as shown on Fig. 5. The resulting embankment seems to offer no advantage over the Hardy core-wall type, as to stability, economy, or tightness; while the rather high seepage loss approaches economic importance. The borrow-pit material at the Bridgewater Dam was only slightly finer than that at the Hardy Dam, and the

resulting core lies well within the "satisfactory" zone of core limits. This would seem to indicate that similar results could have been obtained at Hardy Dam by semi-hydraulic methods. There is the difference, however, that the glacial outwash material has been leached of all alkali content that is present in normal soils, which induces a colloidal tightening not apparent in a physical fineness anaylsis. Furthermore, the usual semi-hydraulic methods used at Bridgewater Dam are not possible because of the cost, the first requirement being not only better but cheaper embankments. However, the possibility, recently developed, of alkali treatment for the up-stream embankment slope, is of interest and should be carefully investigated.

To construct an impervious earth core dam under these typical glacial conditions evidently requires that additional fines be supplied either by pulverizing mudstone or by borrowing glacial till clays from morainal deposits which usually are much farther from sites than the sand borrow-pits. Either of these alternatives would complicate the operations and would add to the cost of a hydraulic-fill dam as usually built. Apparently, such alternatives never cost less than 60 cents per yd in place, in addition to rip-rap, toe drainage, and all other accompanying charges.

Proposals to dredge and pump material from floating equipment, giving access to fines in river beds and adjacent flats, have been advanced with extravagant claims. Probably this would be acceptable structurally, but when reduced to cost figures this method has never been able to compete with other methods.

The Michigan type of sand dam for structures of low head, with thin concrete core-wall, was originally devised to fit the peculiar local conditions most economically. A few have been built of moderate heads without core-walls, but with some serious failures and generally with unsatisfactory operating histories. The core-wall is constructed in sections and by vertical lifts as the fill advances around it. This wall is designed in sections about 20 ft square, well reinforced, with rods extending through the joints. This reinforcement is designed for flexure, and the joint is provided with sheet-metal water-stops. The wall is quite flexible and becomes adjusted to the fill on each side, which, in turn, ordinarily cannot be carried more than 5 ft higher on the high side. The embankment is constructed by filling both sides of the wall at one time, favoring either side as necessary in order to keep the wall aligned. The top also serves as a splash-wall; it eliminates the need of rip-rap, stops burrowing animals, and, to a limited extent, provides a self-supporting barrier which bridges any small washes and erosion until discovered and repaired.

The sand-fill is usually placed by the most economical means in each particular case. No special care is observed in sorting material as to size, but all fill is carefully washed to place with water. This requires moving the dry dumped material by high-pressure hose streams, and confining the flow inside of low shovel-built dikes that conform to the embankment section. Fill material has been transported from pit to section by gravity sluicing or "ground sluicing", wherever possible, this being the most economical of any method. Drag scrapers, mounted on tracks, as well as other forms of con-

veyors, are economical when topography and quantity justify their initial cost. Hauling material by train and dumping it from a trestle is the most flexible method although it is usually more expensive; but any of these methods, and usually some combination, has been used to place embankment material for a large number of developments at a total cost of not more than 30 cents per yd, and including the core-wall the cost does not exceed 35 cents. Earth dam construction has recently been discussed at great length in the technical journals, but almost without exception all reference to cost is omitted. Economy is as essential to good engineering as safety.

However, certain disadvantages of this type of core-wall design become more apparent as it is applied to increasingly higher structures. The part up stream from the core-wall merely supports it, adding little to general stability or percolation distance. At low pond stages, it is attacked by waves unless graveled and formed on a flat slope, not less than 1 on 4 as dictated by experience. The core-wall bisecting the embankment complicates and slows up the operation of placing the dirt and, as shown by test wells and piezometers, does not provide the drop in percolation gradient or saturation plane that might be expected. The ideal construction would evidently consist of some practical and economical method of grading the fill material from the finest up stream to the coarsest down stream (since an earth embankment is essentially a filter), protecting the up-stream slope against wave action, and providing it with a relatively impervious membrane having a seal at the bottom and ends of the embankment sufficient to secure the necessary percolation gradient. It appears certain that the same yardage of fill, and the same expenditure, with properly designed details along these lines, will secure a more stable and tight embankment. Engineering literature is replete with designs and adaptations that endeavor to accomplish this arrangement. Rock-fill dams, for locations where their use is economically determined, are now faced with concrete as standard practice.

Making haste slowly toward this end, because of settlement and adjustment problems under Michigan glacial conditions, the core-wall of Hardy Dam was moved 30 ft 6 in. up stream from the usual location and was "topped out" with slope paving to take wave action for any probable draw-down. The splash-wall, slope paving, and core-wall are all jointed and water-stopped. The relatively negligible settlement and improved tightness to date indicate that complete up-stream paving might well supersede the core-wall in future designs, both improving the structure and decreasing the investment. Of course, paving the face with concrete, for tightness of reservoirs and canals, is an accepted standard construction.

This type of earth embankment, paved for water-tightness, is not new; there are several examples of considerable size, and a number of smaller magnitude. As an example of the possible economies, consider the case of a face slab for the embankment of Hardy Dam, varying uniformly from a thickness of 12 in. at the top to 22 in. at the bottom and laid on a slope of 1 on $2\frac{1}{2}$ (which is conservatively flat). This slab would be designed with a 25-ft arch-web pile cut-off at the toe. A compilation of quantities for this design shows a construction saving of \$150 000, an increase in percolation distance

of 28%, and a symmetrical homogeneous embankment section unencumbered by piles and core-wall, for which construction convenience no credit allowance was made. If this type of embankment permitted a saving of 5 cents per yd in the placing of material (which is not unlikely), this would increase the saving by \$75 000. Probably \$200 000 would fairly represent the difference in cost.

About sixteen comparable examples of face-paved earth embankments are in satisfactory operation in the United States. The only essential difference is that they are all built upon supposedly non-yielding foundations; and the material composing the embankment is also supposedly more uniformly stable. Actually, this settlement record now proves that there should be nothing to fear from adopting this same idea for an embankment design such as that of Hardy Dam.

Settlement of the embankment of Hardy Dam was checked at a large number of points referenced by 2-in. pipes, 6 ft long, driven flush as soon as the fill was "topped out." A small but representative number of these points are represented by data in Table 1, as already discussed. Items Nos. 41 to 44, inclusive, Items Nos. 46 to 49, inclusive, and Items Nos. 53 to 57, inclusive, of Table 1, are all on fills of about the same depth, averaging 55 ft. Lack of exact uniformity is due more to variations in the care taken in washing the fill to place with water rather than to variations in the depth of fill. The area adjacent to the power house and directly over the penstocks, represented by the last five points (Items Nos. 53 to 57, Table 1), was given more care, the results showing in the fact that there was only a $\frac{3}{4}$-in. settlement, and that was quite uniform. Items Nos. 50, 51, and 52 are on the crest at a maximum depth of 125 ft, with an average settlement of $1\frac{1}{4}$ in. and a maximum settlement of $1\frac{3}{4}$ in., there having been no measurable increase from the last records shown—May 1, 1931, to the end of 1932. These results are representative not only of experience at Hardy Dam, but on many previous embankments composed of material from outwash plains of Michigan, when properly settled with water. These conditions insure a structure that is much more stable and free from excessive and continued settlement and sloughing than is usually experienced with well-built hydraulic or semi-hydraulic fill dams, of material ordinarily available.[3]

Percolation and drainage, however, are real problems in heavily glaciated regions. Except for its impervious element, every embankment composed of the available sand material is highly pervious, essentially a filter, as before mentioned. The seepage in this part can be predicted and controlled with much greater certainty than the seepage beneath and around the dam. In such regions the river bed is usually the discharge area for large volumes of natural ground-water that makes its way from adjacent higher levels through the deep and porous strata. Many of these streams are fed as much by this unseen flow as by visible tributaries. When a pond is raised, the entire water-table is likewise affected, and the water finds its easiest path of escape around

[3] See, for example, *Engineering News-Record,* November 14, 1929, pp. 769-772, and July 30, 1931, pp. 1047-1048; see, also, Creager and Justin's Hydro-Electric Handbook, Article 122, p. 254, and the accompanying bibliography.

the ends and along the down-stream side of the embankment. Fortunately, this usual soil formation can sustain a relatively high velocity of outflow without erosion and the springy areas of the natural surface, ordinarily, can be cared for permanently by applying gravel blankets and leading the outflow away in open ditches or tile drains.

The embankment itself should be under-drained to prevent percolation from sloughing away the lower part of the down-stream face and to increase general stability. Usually, this is done by laying bell-and-spigot vitrified sewer tile, with its non-cemented joints packed with marsh hay and protected by heavy paper or felt. The drain is laid in a V-shaped form similar to a pig trough, to preserve its alignment, and is back-filled with gravel. A grid of this construction, laid over about 25% of the total base thickness (or half way in to the normal core-wall location), has been uniformly successful in conducting away all seepage both through and under the dam. It is laid slightly above tail-water elevation so that it will wash itself clean, and preferably on a 1% grade, a sharp gradient being considered as undesirable as one that is too flat. At Hardy Dam a more generous down-stream trench drain was provided by filling the lower part of a dirt trestle, properly lagged with poles, with cobble-stone, out of which 12-in. tile headers projected at 100-ft spacing. This is indicated on Fig. 1. All drain outlets are preferably placed above tail-water, and are arranged separately and visibly for observation and measurement.

The seepage is highly variable as would be expected with the random and heterogeneous arrangement of glacial drift. Although the movement of water through soil has been determined with reasonable precision for any known condition, the haphazard occurrence of a glacial deposit which has been worked and reworked any number of times makes it practically impossible to apply the theory with any dependability. Providing for the maximum, based upon experience, seems to be the only certain method; and providing for it by laying the drainage system before the embankment fill is made—rather than fighting into existing seepage to lay drains—has also been proved, by bitter experience, to be not only safer but far cheaper. The seepage can be controlled in a general way by varying the percolation distance, which is the shortest water route from head-water to point of escape. After safety requirements are met, this control is an economic problem, because the loss of water is frequently appreciable. Part of the core-wall was omitted from a dam of 29-ft head, built in 1916. The initial seepage of 14 sec-ft was about 2% of the stream flow. Natural closure, without silt (the stream is clear), has reduced it to 7 sec-ft in fifteen years. The plant completed in 1926 (71-ft head) had an embankment 4 000 ft long and an initial seepage of 6 sec-ft; in six years this has decreased about one-third. Initial seepage at the Hardy Dam was 6.7 sec-ft, of which 3.7 sec-ft was percolation from the pond and the remainder from ground-water. In six months, it was reduced about 1 sec-ft, which was slightly more than is normally expected, and a smaller initial quantity than was expected, probably due to the deep and carefully driven steel pile cut-off. Since waters in glacial areas are normally clear, not much help from silting is ever realized. The effect of core-wall and pile

cut-off is pronounced, as proved by the first case cited, and similar experience elsewhere. An impervious element in and below the up-stream part of an embankment seems not only necessary for safety, but is economically justified. In one case in which it was omitted, the leakage was about 20 sec-ft, or more than 10% of the stream flow in dry weather. The embankment was as high as that of Hardy Dam, one-third as long, and was built by semi-hydraulic methods.

In most cases no close distinction can be made between percolation through the dam from the pond, and general ground-water seepage. In mid-summer and mid-winter, temperatures will give a fair clue, since ground-water in Northern Michigan varies within the narrow limits of 42° to 46° F. It is usually slightly harder (in some cases, considerably harder), and often there is a slight color difference.

The hydraulic gradient, or plane of saturation, through these sand embankments is relatively high in the structure, and is marked by a separation of dry and saturated material that is quite definitely defined. It varies over quite a range in any vertical plane parallel with the axis of the dam. If the fill material were truly graded and of known size this seepage surface would presumably be regular, uniform, and determinate by analytical methods. Any such gradation of the wide range of random occurrence in economically available borrow-pits has seemed impossible to date, although, admittedly, it would be desirable theoretically. Any simple means to this end would greatly advance the art under these local conditions. Consequently, the saturation line as shown on Fig. 1 merely represents an average of many experiments with Pitot tube and open test pipes in a number of embankments. Sometimes, the core-wall lowers the saturation plane disappointingly little; elsewhere, under seemingly identical conditions, it is quite effective. Experience has indicated a safe generalization for this form and composition of embankment, namely, this plane remains below a 1 on 5 slope from head-water to tail-water; that is, the total travel of water around and under all cut-offs to the nearest point of escape on an embankment face or at tail-water should be at least five times the static head at the point in question. Embankments thus designed, without under-drainage, occasionally have a few damp or possibly wet spots at the toe.

The drains at the Hardy Dam reduce this percolation factor to 4.25, but the filter bed arrangement around the drain heads restrains the material effectively, as compared with a free surface. This factor of 5 is sometimes reduced safely; sometimes it may not be reduced. Many old dams in Michigan, built with little or no attention to design, have stood for years with a factor of 4, and sometimes as low as 3. The critical point usually occurs under the power house or spillway. Many of them have "blown out" from time to time, and usually upon investigation a spot in which the percolation factor is weak is shown by the evidence. A few dams without core-walls have been built with lower factors, usually with serious seepage intersecting the down-stream face. One 30-ft head so built blew out through the base of the embankment, with serious results. Any designer using a factor of less

than 5 to free surface percolation would seem to be lessening the factor of safety, which should be protected in a dam, of all structures. Finer materials require higher factors. Several notable examples in the United States were designed with percolation factors of 8 or 10. In India, factors of 12 to 15 are common, all referring to relatively and uniformly pervious structures as distinguished from dams with dense and impervious puddled cores.

Surface protection on these sand embankments is a problem. The material is so sterile that even weeds will scarcely take root. If the slope is unprotected, wind carries away the fine sand until the remaining accumulation of coarser particles and small pebbles provides protective "rip-rap." The effect of rain is not as serious on this type of embankment as on clay surfaces, because the sand absorbs rain like blotting paper. On a long slope, however, rivulets begin to cut channels at the foot at times of heavy downpour, which erosion may then grow rapidly in size and seriousness, and must be cared for until a protective cover is grown. Berms spaced 20 to 30 ft vertically have been used, but they are expensive to build and require drainage maintenance. Slopes can be seeded by first spreading several inches of soil; but the cost is considerable during the first year for maintenance and care until a sod has taken firm hold. Seeding with a mixture of "red top" with alsike clover and some rye, seems to be the most successful method of establishing a cover. In the autumn winter wheat may be used. To plant "quack" grass (joint grass) is unlawful in Michigan, but there is no better seed for this purpose. However, relative costs of seeding and sodding at the Hardy Dam proved that under these adverse seeding and maintenance conditions, it was much more satisfactory from every point of view to place sod on the face. An abandoned meadow nearly a mile distant furnished the sod, which was cut 3 in. thick by machine, laid directly upon the sand, watered about two weeks, and apparently never stopped growing. Three acres were sodded at a total cost of slightly less than 2 cents per sq ft. The Michigan State Highway Department has established standards for sodding heavy cut-and-fill slopes, apparently for the same reason.

Design of Foundation Structures

Structures properly designed for construction on glacial deposits have proved stable and dependable practically without exception. Those few dams, general buildings, grain elevators, and similar heavy loads that have not secured adequate support on these formations constitute an extremely small fraction of the aggregate, and they indicate (as can usually be proved), that errors have been made in design due to a lack of understanding of the foundation material in question. Undue and disproportionate emphasis on the failures has established an unfair prejudice against it. The Biblical injunction against sand foundations applies to the stated lack of foresight that high water was to be expected. When they have had a clear conception of the fundamentals, adequate experience with glacial deposits, and a wholesome respect for peculiarities and conditions not always evident, engineers have achieved a high percentage of success in building on such foundations.

Settlement should be expected and must be provided for in the design. Limiting this broad subject to the still broad one of dam construction, the problem is usually further complicated by saturated foundation materials and horizontal pressures that induce a variable, and more difficult, distribution of loads. Design theory is largely empirical, being based on experience with the same or similar material elsewhere, and on the results of soil-loading tests, which likewise must be interpreted largely by experience. Relatively few precisely measured records have been kept and made available. In addition to the data herein given, a complete and excellent settlement record of Sherman Island Dam, on the Upper Hudson River in New York State, has been recorded[4] by H. de B. Parsons, M. Am. Soc. C. E. Numerous settlement records of embankments permit no separation of foundation settlement from consolidation of fill; but they appear to be largely or wholly the latter.

The extent and nature of settlement can be predicted with fair success, from boring samples, properly taken and preserved. Delays, failures, and great additional expense have been due to unreliable borings. For example, wash-borings, as usually taken, wash out the fines, disturb and re-arrange the material sampled, and mix it with an excess of water. The test-hole casing should be as large as essential data require and economy permits. An open-end sampling pipe should be used, as large as can be handled inside the casing. At the lower end it should have a short screwed section not more than 1 ft long, with a cutting-edge as sharp as can be maintained. After the casing has been cleaned, by the usual chopping and pumping operation, as near to its lower end as the material permits (but not beyond), and has been bailed out this dry sampling pipe should then be driven into the dry and undisturbed material well beyond the lower end of the casing. The pipe is then pulled and the sample plug is removed from the nipple point as carefully as possible. It is placed at once in a glass jar with a tight cover and properly identified. Casing is then driven a distance determined by the sampling interval, and the process repeated. Many jobs must be prospected with the aid of a well driller and his outfit. Without instruction and inspection, as herein outlined, his record is usually unreliable and sometimes dangerously misleading. On the other hand, after the well driller becomes familiar with this method, the additional labor required ordinarily increases the cost of prospecting less than 25 per cent.

After the foundation material has been so sampled to a depth equal to the head contemplated, a visual examination usually indicates the settlement conditions to be met in the design. More precise methods would be justified in more consistent and continuous deposits, but glacial drift is almost without exception so irregular in its characteristics that the worst, and not an average or general, condition must be met.

Mudstone, gravel, or coarse sand alone have proved suitable for plants up to 100-ft heads, and are desirable, in the order named, for construction reasons. Ordinary stratified mixtures of these materials have consistently resulted in settlements of 1 in. per 5 000 lb of gross load uniformly distributed over large areas. At the Sherman Island Dam previously mentioned a uni-

[4] *Transactions*, Am. Soc. C. E., Vol. 88 (1925), pp. 1257-1292.

form load on similar material, slightly less than 5 000 lb per sq ft, indicated a final settlement of 1.4 in. Mr. Parsons explains, however, that the reference points were on the top runway of the slab-and-buttress concrete structure, 80 ft above the foundation, and, hence, included any elastic adjustment within the structure itself. At both the Hardy Dam and the Sherman Island Dam the settlement records can be adjusted to a gentle easy curve proportional to loads within the range of use (as much as 13 000 lb per sq ft at the Hardy Dam).

Fine sand, grading down to quicksand, is treacherous of course. When it is saturated and when it has an excess of active ground-water, the ratio of pressure induced at right angles may increase to unity. Ordinarily (and preferably to be avoided), foundations on this material, and for moderate loading, can be built and prevented from settling if the bearing area and material can be confined to prevent lateral escape. Thus, as previously mentioned, at a dam completed in 1923, a 20-ft active quicksand stratum, beneath a 25-ft surface of ordinary sand and gravel, was confined by steel sheet-piling, and the enclosed area supports a uniform load of nearly 5 tons per sq ft, on round timber piling. Each foundation condition and design of this kind is a different problem not subject to generalization.

Soft clays and mud, silt, peat, and muck are frequently and generally found throughout the glaciated area. Moderate, quiescent loads can be floated on these materials, but settlement seldom follows a well-defined or dependable program, except that it is likely to continue indefinitely. Most of the foundation trouble in this region has been in connection with this class of material. Highway and railway fills sometimes settle for years, raising billows along each side, and this settlement continues until the unstable material is displaced. A common method of handling fills on such material is to place a part or the bulk of it and then blast the soft layers beneath. The shock of the blast and the superimposed load serves to squeeze out the soft material quickly and to provide a bed for the good material immediately. To construct any foundation on this class of material would be disastrous for a dam of any magnitude, not only as a matter of bearing, but also as regards sliding.

Substructures on sand, gravel, and mudstone should be founded as carefully as possible on undisturbed material, to avoid localized differential settlement. Thorough under-drainage, as previously described, is usually a necessary construction measure, despite the fact that, sometimes, this is not desirable from the standpoint of design. Percolation factors and bearing intensities must be adjusted accordingly. Substructures should be designed as floating units. Adjoining units must be detailed for relative motion considerably in excess of that expected. All changes of base pressure should be accomplished gradually. Base pressure at points adjoining earth fills should be approximately the same as the load exerted by the fill itself, to avoid a plane of rupture due to differential settlement, which is a common cause of failure. The danger of sliding is usually provided against easily, by providing cross-trenches on the bottom of the base pad. Percolation along transverse planes of contact between fill and concrete is blocked by a generous use of "fin" walls.

The foregoing requirements were applied to the Hardy Dam by designing the substructure of the power house and intake tower as completely self-contained, rigid boxes. Although placed in a number of different "pours," each construction "run" is so thoroughly tied into the entire structure, that the whole is virtually a boat, and by actual precise levels, this and other similar substructures actually tilt with each successive "pour." By careful arrangement of the order of "pours" it is possible approximately to hold an "even keel" so as to favor points of joining adjacent structures. The penstocks were designed and built in 30-ft sections, joints being detailed to permit settlement and adjustment by special provisions in both steel and concrete. Soft pine form lumber was left embedded around the top half of joints of concrete, so that it would compress with settlement. Steel liners were fabricated and assembled so that considerable angular adjustment could take place at each joint. The amplitude of possible movement was based on a possible settlement of 5 ft under the crest of the dam, although only 6 in. was the maximum expected, and about 3 in. was all that was actually experienced. A 6-in. upward camber was put in the penstock grade and unwatering the penstock affords a check that only half has been removed by settlement of about 3 in. as recorded. Precise levels the full length of each of the three penstocks show easy and almost identical curves as indicated by the average in Fig. 1. Concrete joints in the penstock were matched to prevent any tendency to offset; no such tendency has been indicated. The core-wall was notched by tongue and groove into the penstock shell to permit any relative movement (of which there has been $\frac{1}{2}$ in.) and still maintain water-tightness. Although the steel "liners" in the penstock were heavily "ribbed" with angle stiffeners and otherwise anchored into the concrete shells, no attempt was made to make them absolutely water-tight. In fact, the parts up stream from the core-wall were perforated with $\frac{1}{4}$-in. holes on about 8-ft centers to relieve any water pressure between "liner" and shells. While these precautions are somewhat unusual, and even though jointed, adjustable penstock sections are not general, no distress has been evident to date (1933), as distinguished from experience elsewhere with rigid, non-adjustable, and unvented designs.

To require that a power-house structure be designed to insure all pours being knit together into a rigid self-supporting whole, imposes a much more severe condition than if the foundation was rock. Loads must also be arranged so that the combined load and thrust will intersect the base near the center, in order to preserve plumb and level conditions as far as possible. The substructure of the power house at the Hardy Dam was determinate as to its loads and thrust, but the effect of the adjacent embankment loading was indeterminate. Its estimated value was actually exceeded, with the result that the structure tilted up stream $\frac{3}{8}$ in. in a width of 106 ft. This is less than is frequently experienced with power-house structure of the open penstock type, built entirely through the embankment.

This tilting during the construction of practically all substructures on Michigan rivers is of no disadvantage, except as it concerns the usual methods of erecting power-house machinery. Erectors normally proceed by plumb and

level methods. In fact, at first any other program seems impossible to them. Turbine discharge rings are always set level; then speed-rings are set to them, or sometimes *vice versa*. Several months elapse before the substructure is complete around the penstock, scroll case, and pit liner, ready for the turbine assembly. By that time the final weight has largely been added and its distribution may be decidedly different from what it was when the discharge ring was set. Consequently, the erector then finds his contact surfaces neither plumb nor level, and he either throws up his hands in disgust, or insists on shimming everything back to plumb. He would not appreciate that still further loading, and consequent adjustment, would change this condition again, before the unit was erected, and would make future dismantling and assembling almost impossible. In time, the erector understands that all parts should be set square and concentric with the speed ring "as is," and that they should be properly match-marked and doweled for future assembly. With many units thus assembled, no operating disadvantages have ever been noticed for the slight amount that a vertical shaft is out of plumb.

Uniform distribution of base-slab loading, over wide draft-tube spans, is often best accomplished by a liberal use of heavy steel beams or open steel trusses embedded in concrete. Embedded steel trusses are used where necessary to secure the end-to-end stiffness of the entire structure. Fortunately, the use of steel in these structures whether reinforcement, structural, or plate, is free from the dangers of corrosive action caused by less pure waters and soils that are not water-washed as is this glacial drift. Experience of thirty years shows practically no rust on steel maintained 5 ft, or more, under water, even without paint or protection of any kind. A greasy scum, apparently of animal life origin, provides a protective coating. At certain seasons when pits are unwatered for inspection the turbine-case castings are literally covered by small white grubs. Structural plates, washers, and bolts, not protected by paint and removed after approximately twenty years, had mill scale intact under this coating.

Round timber piling is used as little as possible for substructure support, and then only when any possible water channels beneath its supported structure are effectively blocked by steel pile cut-off walls. Furthermore, its resistance to lateral thrust is so uncertain that all such thrust must be directed elsewhere. Round pile foundations, however, are widely used for general construction in the glacial drift areas, where material is too soft to support loads on spread footings, which are given preference when suitable. When timber piles are driven below ground-water level (and this would apply practically to the construction of all hydro-electric plants), there is no measurable deterioration, and there is no teredo organism. Logs are still being salvaged commercially from river beds, where they have lain since lumbering operations in the Seventies, and they are structurally as sound as new timber, although the sapwood sometimes is stained. Round piles usually have an ample factor of safety even when loaded to the capacity indicated by the *Engineering News* formula. Any types of sand and gravel afford high skin friction and, usually, driving is best accomplished by jetting, continu-

ously, up to the point of seating. Even in quicksand, the pile is seized rigidly when jetting ceases, and, frequently, it cannot be started again without jetting.

Steel sheet-piles are used generously for cut-off purposes. In fact, without modern piles and pile-driving equipment, dam construction on these glacial deposits would be limited to plants of relatively low heads. Steel pile cut-off walls prove most effective in blocking under-flow, and the heights of structures now seem limited only by the depth of the cut-off that can be driven (at present limited by practical considerations to about 60 ft in sand and gravel). Driving is easier in clay and even in mudstone, but sand packs in the interlock between steel piles with a grip that practically stops driving at a penetration of 60 ft, even with a double-acting steam hammer striking an 11 ft-ton blow. No known method of jetting, and no form of interlock yet devised, will keep that interlock free beyond this limit. Jetting is an essential part of deep driving in these formations, not only to reduce skin friction and to free the interlock, but also to permit drawing an occasional boulder to one side. While not numerous below the accumulation on the river bed itself, there are enough boulders to cause some trouble and concern. Although some engineers have doubts as to the efficiency of steel pile cut-off walls for this reason, results show that an experienced driving foreman can tell when he hits a stone, can guard against "jumped" interlocks, and can detect them quickly enough if he is conscientious. Piles are sometimes pulled back as a test of driving results. In one case of extreme and unwarranted caution, all the piles (only 25 ft long) on a recent dam job in Southern Michigan were pulled, inspected, and re-driven, the cost in this instance being apparently of no concern. A moderate use of such methods, or even the threat of them, will guard against breaks in the cut-off wall. Mandatory open-trench cut-off construction, as on the Bingham Dam, in Maine, would make Michigan projects impossible economically.

Pressure grouting with cement, mud, asphalt, etc., has been tried, but with little success because the grout goes into the more porous openings of least resistance which, almost without exception, are filled with large quantities of ground-water under head. This water is usually flowing with considerable velocity toward some point of escape not far distant. The excavation of grouted areas shows merely a series of discontinuous and random masses that are of little use as a continuous cut-off. Up-stream earth blankets are good, but are scarcely dependable unless backed up by a steel cut-off wall. The combination is practically water-tight, however, because the interlock becomes packed with sand so completely as to plug even its slight clearance. Steel piles are not used for bearing, except in some relatively unimportant position. Under the core-wall and penstocks, a rough wood box form to exclude concrete is placed over the top to provide a slip joint and this form is left in place. The concrete footing of the core-wall secures a grip on each side of a pile, below the top, and keeps a tight joint if the core-wall settles over it. When steel piling is used at, or above, the water line, it is galvanized. When used for revetment walls to maintain river channels, canals, and raceways, the results of this system are good, and economical.

Spillway Requirements

In Lower Michigan, insufficient spillway capacity, as far as known, has never caused the failure of dams of any magnitude or of dams built on the basis of engineering design. Numerous old mill dams have washed out from time to time, from various causes, and while, unfortunately, no distinction is made by the general public, the design of these structures is seldom based on engineering study.

Application of capacity requirements based on engineering study must be modified, of course, for such matters as ice, rubbish, frozen-in gates, etc. Heavy ice forms on power ponds in this latitude, but pond ice has never been known to run on the Muskegon, Manistee, or Au Sable Rivers. Break-up flood flows are insufficient to move it down river, and it melts in place. This would scarcely be true of smaller streams, with smaller ponds, even in the same vicinity. This pond cover effectively prevents the formation of frazil and slush ice. Lumbering rubbish is an aggravating nuisance on these rivers, but with boom control it has never interfered with the opening of spillway gates, although it has interfered with closing them. Free crest spillways would insure against these possibilities, but these have not been required. Gated control, which has decided advantages and economies, has been used almost entirely. Steel Taintor gates have been standardized in a size, 21 to 24 ft wide by 13 ft high, with a capacity of about 4 000 cu ft per sec per gate without head rise. First cost is so moderate, operation is so simple, and maintenance is so low that it is difficult to visualize the necessity for the complicated and expensive roller type which has been installed in many developments.

As far as size and capacity are concerned, Taintor gates have been built and operated with complete success in Northern latitudes, and in ice-laden rivers, in sizes up to 80 ft long and 30 ft high. Their weight is only a fraction of that of the other type, and the setting and hoisting details and equipment are far more simple. When properly designed and detailed, they are entirely determinate in stress analysis, decidedly economical in structural design, as tight as any other gate, free from damage due to spilling débris, and low in maintenance. Means have been developed to prevent their freezing in, and the same measures can be applied to them as easily as to other types. Recent advances in welding have further decreased their cost and improved their tightness. On one hundred spillway gates of this type, and on thirty penstock gates (usually 30 by 30 ft), within the writer's experience, no Taintor gate of this design has failed structurally, or has refused to operate when required. Under-sluices beneath power houses have been used in a few plants. They permit closer and easier pond control for small waste discharges without outdoor spillway operation.

Spillway design on glacial drift is somewhat more difficult than on stronger foundations. The structure must be sealed and provided wth a cut-off wall that is even more conservatively designed than for the embankment, since percolation distance under it is usually shorter than beneath an embankment. Generous use of steel piling and cut-off fins provides these

requirements. By using a buttressed and inclined-slab type of concrete structure, foundation loading is held within bounds. For the higher heads gravity sections cannot be supported, even if they were otherwise desirable. Spilled water energy cannot be turned loose on foundation material, but must be dissipated by stilling pools or by baffles. Outflow velocities must ordinarily be held to a maximum of 4 ft per sec, and kept straight and uniform off the end of the apron. Tumble bays were built at earlier plants to absorb energy. They were designed before model testing was developed to its present high state, and many were only partly successful. Submerged weirs were used later to take the shock of discharge and to create standing waves. These were tested by means of models and were more satisfactory. More recently dentated sills (not necessarily of patented design) have been used for the same purpose and with most satisfactory results. A dentated sill only 46 ft from the free discharge end of a 6-ft spill tube, operating under a full head of 150 ft, destroys the energy so effectively that sand collects almost up to the level of the apron slab between the sills. Dentated sill designs, developed and tested by models, have greatly improved spillway conditions on soil foundations.

Further study and model testing are likely to develop some arrangement similar to that shown in Fig. 6. The construction trestles in this section are placed in echelon and are built to a height and spacing that will fit the topography and will permit progress from the ends toward the center, using cross-dikes. The pit-run sand for the embankment is delivered from the trestles and sluiced up stream on a slope of about 1 on 8, thus forming a blanket of fines over the base and grading the finest material up stream throughout the height of the embankment.

For complete segregation of the washed portions (practically attained by washing "to place"), the fineness gradation would be about as shown in Fig. 6. The interlocked steel sheet-piles at the up-stream toe would be of a length dependent upon the subsurface formation. In general, they would be long enough to insure a percolation factor of 5 to drains. These piles are driven after the fill has reached the elevation of the top of the piles, and the top is cast into the paving slab as shown.

The up-stream slope is diked and vented during construction and the excess material over the neat grade is placed in a blanket to form a buttress and seal at the toe.

The concrete paving is laid in articulated tongue-and-groove slabs, well reinforced, and provided with crimped metal water-stops in all joints. The slab is well keyed into the slope to prevent sliding.

Conclusions

On the basis of the foregoing arguments the writer offers the following conclusions for discussion:

(1) Coarse sand, gravel, or glacial mudstone river-bottom formations support loads satisfactorily if such loading is graduated gently to a maximum of 13 000 lb per sq ft, with an average allowable settlement of 1 in. per 5 000 lb per sq ft of loading.

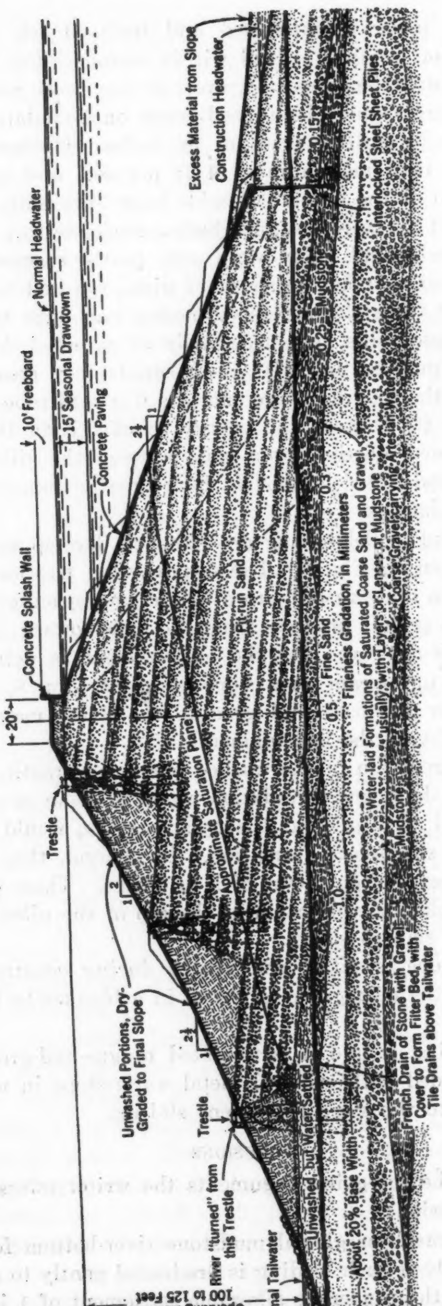

FIG. 6.—IMPROVED SAND EMBANKMENT ON TYPICAL GLACIAL OUTWASH FOUNDATION

(2) The performance of this type of foundation indicates the probability that tests may develop still greater load-carrying ability without excessive settlement or other difficulties.

(3) Settlement is essentially proportional to load and takes place as the load is applied; it practically ceases thereafter.

(4) Sand embankments, composed of river valley and glacial outwash material, settle in permanent position and form when washed to place with water. Any subsequent consolidation is too slight to effect up-stream paving that has been properly detailed and laid on a 1 on $2\frac{1}{2}$ slope.

(5) This available embankment material is deficient in fines for the usual impervious earth core designs, and must be supplemented by some impervious element forming a part of the embankment. Alkali treatment should be investigated.

(6) Embankments of this material, on good sand and gravel foundation, can probably be constructed satisfactorily for heads in excess of 100 ft, by re-arranging the foundation cut-off and impervious members to better structural advantage, and with improved economy.

(7) Penstock structures properly designed in articulated sections for settlement may be built safely through embankments and on foundations as herein described.

(8) Spillway capacities for a 1 000-year flood increased by 20%, are ample.

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PAPERS

IMPROVED TYPE OF FLOW METER FOR HYDRAULIC TURBINES

BY IREAL A. WINTER[1], ASSOC. M. AM. SOC. C. E.

SYNOPSIS

The operation of the modern hydro-electric power plant for high efficiency necessitates the determination of the actual turbine and generator performance curves. Such curves can now be determined to a high degree of accuracy by rating the turbine water input by either the Allen or the Gibson method. These ratings are usually in the nature of a turbine efficiency test made to determine whether the manufacturer's guaranties have been met, or to check the efficiency of old installations. As the performance curves thus established apply to only one head and one plant-loading condition, its utility as a means of presenting operating data has been considered secondary. This attitude has been justifiable, to some extent, as no generally applicable means have been available for rating permanently, a device that would indicate the turbine discharge independent of the head and load on the plant.

The necessity of using, to the fullest extent, the rate of flow data obtained during the turbine efficiency test, led to the development of the Winter-Kennedy principle for obtaining a permanent plant discharge record. The principle utilized is based on fundamental laws of flow, enabling the prediction of performance before installation. Pressure differences existing in the conventional turbine scroll case serve as the prime movers and, therefore, no loss of pressure head, change in direction of flow, or turbulence, is set up in the water passages of the turbine. The means utilized are flush piezometers located on the inner and outer surfaces of the scroll case.

The writer describes the fundamentals of this new method of determining the turbine discharge, and discusses, at some length, the results obtained on models tested in the laboratory and actual plant installations.

NOTE.—Discussion on this paper will be closed in August, 1933, *Proceedings*.

[1] Birmingham, Ala.

Introduction

The increased efficiency of present-day hydraulic turbines and the added refinements to power plants representing large financial investments, suggests the need of accurate knowledge of the water input and power output.

When the turbine is driving an electrical generator, the determination of the power output is relatively a simple matter. Electrical metering instruments, capable of a high degree of accuracy, are available for every type of service. The measurement of the rate of flow of large quantities of water in the field, however, is still recognized by hydraulic engineers as one of the most difficult of engineering problems.

Methods of rating water passages have been developed by American engineers, to a point where reliable and consistent results are obtained, covering a wide range of conditions. The methods developed by C. M. Allen, M. Am. Soc. C. E. (salt-velocity method[2]), and N. R. Gibson, M. Am. Soc. C. E. (time-pressure method[3]), have met with wide approval. The momentary rate of flow may be determined by both these methods, leaving the problem of permanent plant recording to be solved successfully.

Early attempts to solve the combined problem of rate of flow and plant recording were made, using the Venturi tube. This type of meter has proved practical for high-head impulse installations where ideal pipe-line conditions are to be had.

When attempts were made to adapt Venturi tubes to plants of relatively low head and short penstock, the expense of the tube and loss in pressure head made them economically unsound. It was discovered that Venturi tube characteristics were subject to considerable variations when departures from ideal conditions were encountered. The meter coefficients were found to vary as much as 10% from the ideal, and square law ratios did not exist.[4]

Another attempt to combine rate of flow and plant recording was made with the pitometer. This instrument proved unsatisfactory because it was subject to plugging by trash or because it was knocked out of position by solids carried in the water stream.

Many successful installations of equipment for obtaining permanent plant records have been made by placing piezometers in converging sections of water passages or on elbows in the penstock. As these devices are dependent upon plant conditions, they do not have general application.

It is significant to note that all recording devices for use with plants of large turbine discharge and low head must be rated in the field by some independent method of water measurement. This is also true of the method developed by Mr. A. M. Kennedy and the writer. The advantages of the improved type of flow meter are that it has general application, and its performance can be predicted with sufficient accuracy to enable the preparation of designs before the plant is constructed.

[2] *Transactions*, A. S. M. E., Vol. 45, No. 1902, p. 285.
[3] *Loc. cit.*, No. 1903, p. 343.
[4] Serial Report, Publication 278-34, National Electric Light Assoc., pp. 6-7.

Scroll-Case Design

An understanding of the basis of designing scroll cases is necessary in considering the operation of the differential pressure taps. There are three general methods of arriving at the progressive areas of the scroll, namely, the accelerating velocity method, in which the area progressively decreases relative to the increment of water discharged into the turbine; the constant velocity method, in which the area decreases directly as the increment of flow into the turbine; and the decreasing velocity method, in which the net area of the scroll case is progressively greater than the increment of flow into the turbine.

The three methods involved are based on the fundamental law of conservation of angular momentum, or the law of the free spiral and circular vortices. Only in the accelerating scroll case of the free spiral vortex design does the center of the turbine and vortex coincide.

For structural considerations, a scroll case is not the path following the stream lines of a vortex. Investigation of the flow within the scroll shows a wide range of stream lines into the turbine speed-ring.[5] The discharge varies from a radial flow on the upper and lower surfaces of the scroll adjacent to the speed-ring, to an angular flow greater than the angle of the speed-ring stay-vane at the center line of the distributor.

Concrete scrolls of rectangular section and plate-steel scrolls of circular section are of similar design. The difference in shape is due to structural rather than to hydraulic requirements. The baffles in plate-steel scrolls are located substantially at a point 90° to the transverse counterline of the turbine. In the case of the concrete scroll, considerable latitude is taken in locating the baffle. Efficient designs have been executed with placement at points ranging from 90° to the transverse center line of the turbine, to a point substantially on the down-stream center line, covering an arc of 270 degrees. In all cases the remaining part of the true scroll passage is designed according to one of the three methods outlined.

Notation

The following notation is used throughout the paper:

A = area of scroll case at piezometer section.

C = coefficient of deflection.

H = head: H_e = effective pressure head on turbine; H_{V1} = velocity head corresponding to tangential velocity component, V_1; H_{V2} = velocity head corresponding to V_2; H_t = effective pressure head on turbine at time of test; and H_c = a common pressure head selected for convenience of comparing test data.

P = differential pressure between piezometers: P_t = pressure at test head; P_c = pressure at a common head.

Q = discharge: Q_t = discharge through the turbine at time of test; Q_c = discharge at a common pressure head; and Q_n = net expected maximum discharge through the piezometer section.

R = a distance measured from the center of the turbine: R_1, R_2, etc. = distances to piezometers; and R_x = distance to center of gravity of piezometer section.

[5] *Transactions*, A. S. M. E., Vol. 53, No. 13, HYD-53-4, p.32.

V = velocity: V_1, V_2, etc. = tangential velocity components at piezometers; and V_a = mean velocity at center of gravity of piezometer section.
g = acceleration due to gravity.
h = head: h_v = any velocity head; and h_f = any friction head.
k = experimental constant.
m = mass of water flowing at any point in the piezometer section.
n = logarithmic exponent.

FLOW METER DESIGN

As previously pointed out, the flow lines within the scroll case do not follow definite directions, but are subject to considerable variation. This combination of flows invites an investigation as to their possible effect upon pressures at various points across the section. The following laws of flow are significant:

(*a*) For radiating currents, the pressure head at any point distant from the center is a function of,

$$R_1 V_1 = R_2 V_2 \quad \text{.........................(1)}$$

(*b*) Likewise, for a revolving mass of water in which the stream lines are concentric circles and the total pressure head for each stream line is the same, the pressure head at any distance is a function of Equation (1).

(*c*) Furthermore, for a revolving mass of water having a radiating flow combined with a circular flow, the pressure head at any distance from the center line of the turbine is a function of Equation (1), for both components of flow, and with positive or negative acceleration.

(*d*) If no change in direction or relative magnitude of flow takes place within the scroll case, the differential pressure, P, existing between any two points is found to be,

$$P = \frac{V^2_1}{2g} - \frac{V^2_2}{2g} \quad \text{....................(2)}$$

Therefore, the flow, Q_n, through the scroll at the piezometer section is a function of the square root of the differential pressure, P. Laboratory and plant tests covering a wide range of conditions confirm these conclusions.

In designing an installation of scroll-case differential-pressure taps, the relation, $R_1 V_1 = R_2 V_2$, is used as a basis. The velocities, V_1 and V_2, are considered as tangential components of the absolute velocity, and the radii, R_1 and R_2, are their distances from the center of the turbine. This assumption is necessary as the absolute direction of the flow is unknown. Therefore, Equation (2) must take the form of,

$$P = C\left(\frac{V^2_1}{2g} - \frac{V^2_2}{2g}\right) \quad \text{..................(3)}$$

The quantity of water flowing at the section selected for trial, is determined by assuming the entrance edge of the speed-ring stay-vanes as the orifice of discharge from the scroll case (see Fig. 1). The total arc of the speed-ring orifice, through which the total discharge, Q, flows, is deter-

mined by deducting the thickness of the scroll baffle on a line with the circumference of the stay-vane tip of the speed-ring. The net quantity, Q_n, flowing past the piezometer section is taken as the percentage of orifice circumference past that section. This establishes the net flow to be used in the calculations. The quantity, Q, when used as a basis of design, is always

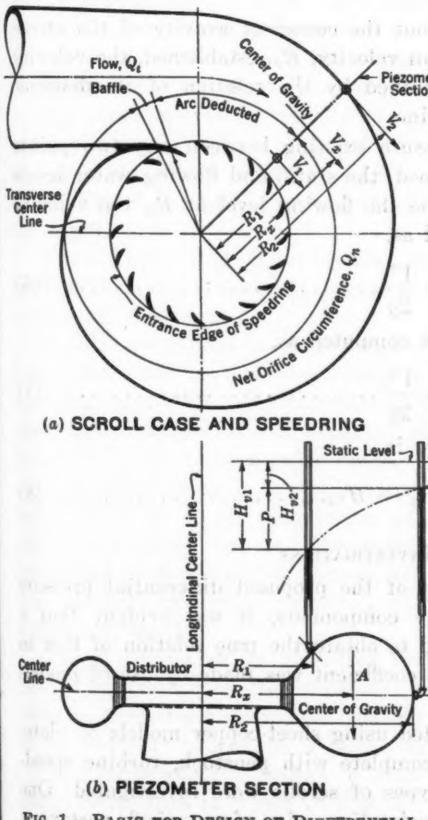


FIG. 1.—BASIS FOR DESIGN OF DIFFERENTIAL PRESSURE TAPS

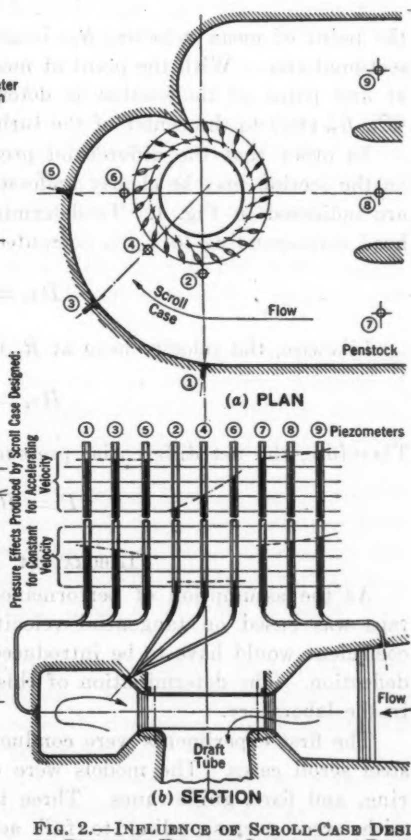


FIG. 2.—INFLUENCE OF SCROLL-CASE DESIGN ON PRESSURE EFFECTS OBTAINED BY DIFFERENTIAL PRESSURE TAPS

taken as the maximum expected discharge through the turbine. With the net flow and the area of cross-section of the scroll case known, the mean velocity is,

$$V_x = \frac{Q_n}{A} \quad \text{.............................(4)}$$

As shown by the relation, $R_1 V_1 = R_x V_x$, the increment velocities vary inversely as the radii. Hence, the velocity curve from the entrance of the speed-ring to the outer wall of the scroll case takes the form, $y = cx^n$, with the exponent, n, at unity.

For stability of flow, the revolving mass of water must have a common center of gravity. This common center of gravity is assumed to be the center of gravity of the cross-sectional area of the scroll-case section. Then, by the law of constant moment of angular momentum,

$$R_1\, m\, V_1 = R_2\, m\, V_2 \quad \text{.......................}(5)$$

the point of mean velocity, R_x, is about the center of gravity of the cross-sectional area. With the point of mean velocity, R_x, established, the velocity at any point of the section is determined by the relation of its distance (R_1, R_2, etc.) to the center of the turbine.

In order that the differential pressure existing between any two points on the section may be clearly understood, the static and flowing water levels are indicated in Fig. 1. To determine the flowing level at R_1, the velocity head corresponding to V_1, is computed as,

$$Hv_1 = \frac{V_1^2}{2g} \quad \text{............................}(6)$$

Likewise, the velocity head at R_2 is computed as,

$$Hv_2 = \frac{V_2^2}{2g} \quad \text{............................}(7)$$

Therefore, the net differential pressure is,

$$P = Hv_1 - Hv_2 \quad \text{.......................}(8)$$

Laboratory Investigations

As the assumption of performance of the proposed differential pressure taps was based on tangential velocity components, it was evident that a coefficient would have to be introduced to obtain the true relation of flow to deflection. The determination of this coefficient was made by use of models in the laboratory.

The first experiments were conducted using sheet-copper models of plate-steel scroll cases. The models were complete with penstock, turbine speed-ring, and fixed guide-vanes. Three types of scrolls were investigated: One with areas corresponding to full acceleration of a free spiral vortex; a second, with acceleration comprising the spiral and circular vortex; and the third, with areas corresponding to negative acceleration. Piezometer sections were established at quarter-points in the cases.

The total rate of flow as determined by pressure readings for the various sections of all three scroll cases was found to be, approximately,

$$Q = k\, P^{0.500} \quad \text{.........................}(9)$$

The coefficient, C, in Equation (3), was found to vary with the ratio of height of scroll case at R_x, to height of turbine guide-vanes—the smaller the ratio, the larger the coefficient. The absolute value of C ranged from 0.75 at the entrance to 1.25 at the last quarter of the scroll.

Subsequent tests were made on models of a concrete scroll case in connection with a 16-in. runner, operating under a head of 10 ft. These results proved more interesting as greater deflections were obtained and the influence of the change of angle of turbine gates could be observed.

The characteristics of pressure differences observed in the two types of scrolls are shown in Fig. 2. It is significant to note that for the case with constant velocity areas, the outer, or high-pressure, piezometers (Nos. 1, 3, and 5) record substantially the same elevation. The inner, or low-pressure, piezometers (Nos. 2, 4, and 6), vary as a function of R_x.

For the accelerating velocity case, the characteristics are reversed. The outer piezometers (Nos. 1, 3, and 5), varied substantially as the change in radius of the center of gravity of the scroll section, while the inner piezometers (Nos. 2, 4, and 6), registered equal elevations at various points around the scroll.

These interesting facts suggest that a flow is obtained more nearly approaching the vortex principle than is indicated by the flow lines on the surface of the scroll.

Similar coefficient characteristics as observed in the experiments on the 6-in., plate-steel models were found to exist for the rectangular sections of concrete cases. The absolute value of C is higher for the rectangular section, when based on the ratio of height of the scroll case at the center of gravity, to height of the turbine guide-vanes.

Piezometers Nos. 7, 8, and 9, shown in Fig. 2, are included as interesting information as to the conversion of centrifugal force into pressure head within the scroll proper. Pressures registered by these piezometers are influenced by many outside factors, and a definite relation of flow to deflection does not hold true when compared with each other, or with the pressure units within the scroll.

These differences in pressures over the scroll sections should be more clearly understood by the structural engineer. The eccentric loadings indicated by these tests, introduce bending moments not taken into consideration in the ordinary procedure of designing power-houses.

The influence of a change in head on the turbine on the performance of the pressure taps is shown in Fig. 3. The plotting is for readings made on Piezometers Nos. 1 and 2, as shown in Fig. 2. The test points indicated are for five gate-openings, covering a turbine speed ranging from 75 to 115% of the normal designed head (4 to 7 speeds each, over a range from 84 to 110 rpm.). This curve clearly illustrates that no change in characteristics takes place due to a change in angle of the turbine guide-vane, or relative speed of the runner. This is important as there are three factors tending to produce unstable flow conditions at the speed-ring entrance where the low-pressure tap is usually located.

The distribution of velocities into the speed-ring is not symmetrical or uniform about the center line of the distributor. This is due to the lower section of the speed-ring supplying water to a larger area and shorter path to the turbine runner. As the guide-vanes are closed, the distribution is more symmetrical because the gate-opening becomes the controlling orifice.

A second influence in the distribution of velocity adjacent to the speed-ring is the relative position of the entrance edge of the guide-vane to the discharge tip of the speed-ring stay-vane. Cases have been known in which the change of flow from one side of the speed-ring vane to the other caused a drop of $2\frac{1}{2}\%$ in over-all turbine efficiency. Any pressure tap within the influence of this velocity change would register unstable characteristics.

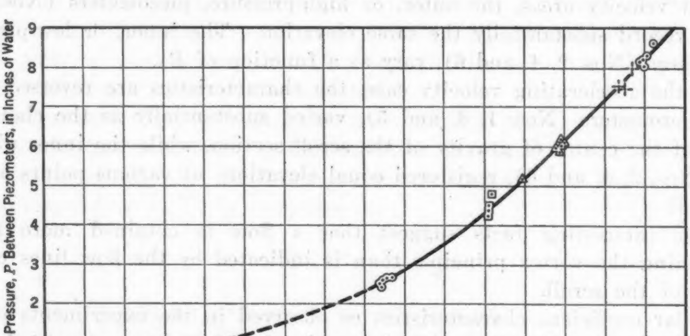


FIG. 3.—DIFFERENTIAL PRESSURES FOR 16-INCH TURBINE MODEL, SHOWING EFFECT OF CHANGE IN GATE-OPENING AND SPEED OF RUNNER

The third factor producing undesirable pressure effects is the constant speed of the runner for all heads. This item is seldom of importance because the head on the turbine rarely ever approaches a value such that a short circuit of flow is likely to take place on the entrance side of the runner.

In all cases tested in the laboratory, the inner piezometers were outside the speed-ring casting. The influence of gate changes and speed of runner could not be found for any combination of conditions.

From the results of laboratory investigations, the following conclusions may be deducted:

First.—For all scroll cases designed according to one of the three methods outlined, the following relation was determined:

$$Q = k\ P^n \dots\dots\dots\dots\dots\dots\dots\dots (10)$$

Second.—The coefficient, C, was determined as varying from 0.75 to 1.25.

Third.—The coefficient, C, is substantially constant when the laws of similitude are applied. This makes possible the utilization of experimental coefficients for locating pressure taps in any scroll case.

Fourth.—The exponent, n, was found to be substantially 0.500.

Fifth.—The low-pressure taps, when properly located with reference to the speed-ring, are not subject to influence by a change in the angle of the turbine guide-vanes, or to a change in head on the turbine.

Flow Meter Application

Plant Installation.—A typical section through an hydro-electric power plant is shown in Fig. 4. An installation of differential pressure taps is shown, diagrammatically, on the down-stream side of the power house. The differential pressures are related to the static forebay pressure. The notation, $h_f + h_v$, indicates the total friction-head and velocity-head losses incidental to the penstock and other intervening water passages leading to the forebay.

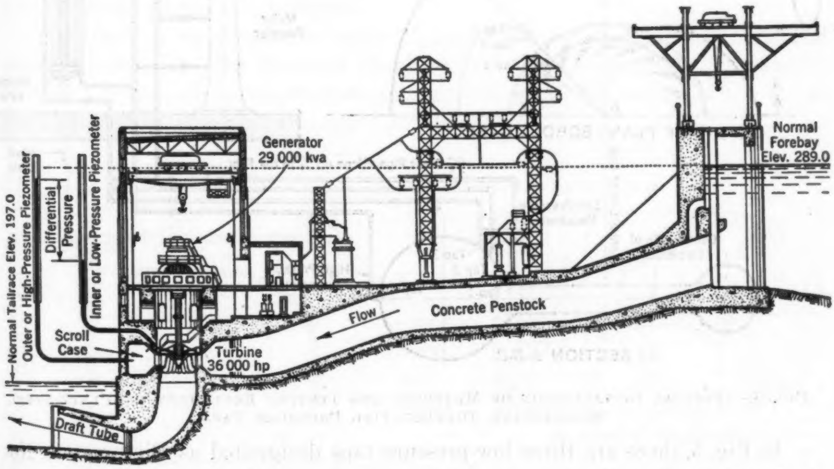


FIG. 4.—TRANSVERSE SECTION THROUGH HYDRO-ELECTRIC PLANT, SHOWING EFFECT PRODUCED BY SCROLL-CASE DIFFERENTIAL PRESSURE TAPS

A typical installation of metering and testing equipment connected to scroll-case differential pressure taps, is shown in Fig. 5. Such a combination gives complete water information for the unit. The mechanical register indicates the rate of flow, in cubic feet per second, draws a chart record showing the change of flow through the turbine, and totalizes the quantity of water passing through the machine, in cubic feet. With these data before him, the operating engineer can determine the efficiency of the unit covering any particular period of time.

The throttling manometer, shown in the illustration, is especially designed for determining the quantity-deflection relation at the time of test. Its several features consist of an engraved scale, graduated in inches and tenths, with the zero of both scales at the bottom. The scales are placed on the center line of the glass tubes, enabling accurate observations on the meniscus of the mercury column. An equalizing valve and suitable air vents provide means of obtaining a clear column of water connecting each leg of the manometer. The throttling valve at the bottom of the U-tube enables the successful dampening of the mercury column without introducing the usual errors inherent with throttling at points in the water line. By use of this manom-

eter, check tests can be made on the performance of the register, and readings on higher or lower pressure taps can be compared with those on the standard taps in case of an emergency.

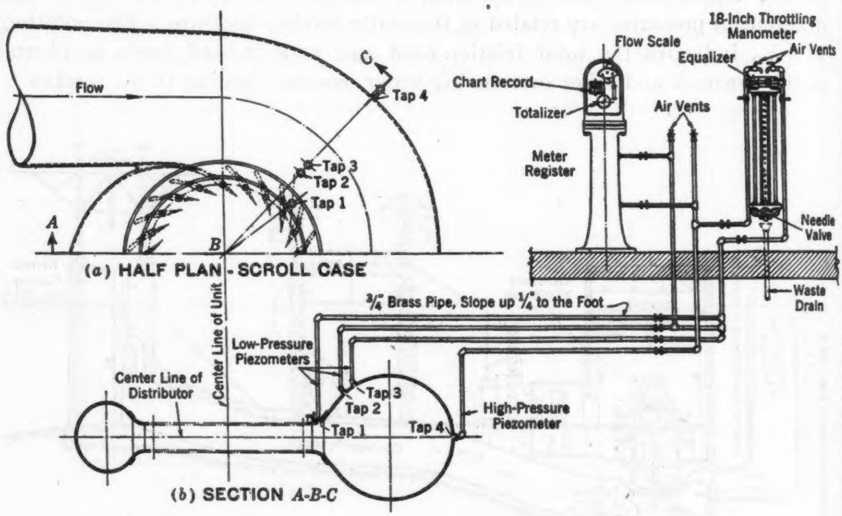


FIG. 5.—TYPICAL INSTALLATION OF METERING AND TESTING EQUIPMENT ON PLATE-STEEL, SCROLL-CASE, DIFFERENTIAL PRESSURE TAPS

In Fig. 5, there are three low-pressure taps designated as Piezometers Nos. 1, 2, and 3. Tap No. 1 is a special connection to obtain a large differential pressure. The performance of this tap is influenced by a secondary factor due to the centrifugal force, in a vertical plane, of the water flowing around the curvature of the speed-ring casting. This secondary force is of prime importance as it can reach a value at high flows for which the quantity-deflection ratio departs from the square law. This is due to the tendency of the water to leap clear of the casting as the velocity increases and the restraining pressure decreases.

Pressure Taps Nos. 2 and 3 are designed as standards to meet the range of deflection of commercial meter registers. In all cases there should be two inner taps so designed that a variation of 20% in flow will produce like pressures. This is necessary, due to a possible increase in expected turbine discharge. If only one tap is installed, the capacity of the register might be exceeded, making necessary the purchase of a special instrument.

The type and method of installing the piezometers within the scroll case are of first importance. Two of the most typical designs are shown in Fig. 6. Fig. 6 (*a*) and Fig. 6 (*b*) illustrate the correct method of fastening piezometers to wood forms and of extending pipes through the concrete. Details of two types of piezometers are shown in Fig. 6 (*c*) and Fig. 6 (*d*). The Type *A* piezometer is for use with the plate-steel scroll case and speed-ring castings. The Type *C* piezometer (for use with a concrete scroll case) is anchored to the forms with three No. 10, flat-head, brass screws, $1\frac{1}{2}$ in. long.

After the forms are stripped, the screws are cut off flush with the face of the brass plate. Thus, after the walls have been cleaned, the drilling of the $\frac{3}{8}$-in. hole is completed so as to make the pressure connection to the $\frac{3}{4}$-in. line.

The reason for drilling the piezometer opening only part way in Type *C* is that cement mortar is likely to run into the piezometer line and plug the passage during the process of pouring the concrete. The completion of the hole after the forms have been removed is a simple matter. By providing the anchorage holes for the brass screws, damage to the piezometers when removing the forms is eliminated.

The Type *A* piezometer is made of monel metal for rust resistance and metallic strength. The threaded shank should never be tapered. Invariably, if it is, the nose of the piezometer will not extend to the inside of the scroll, thus creating a most embarrassing situation.

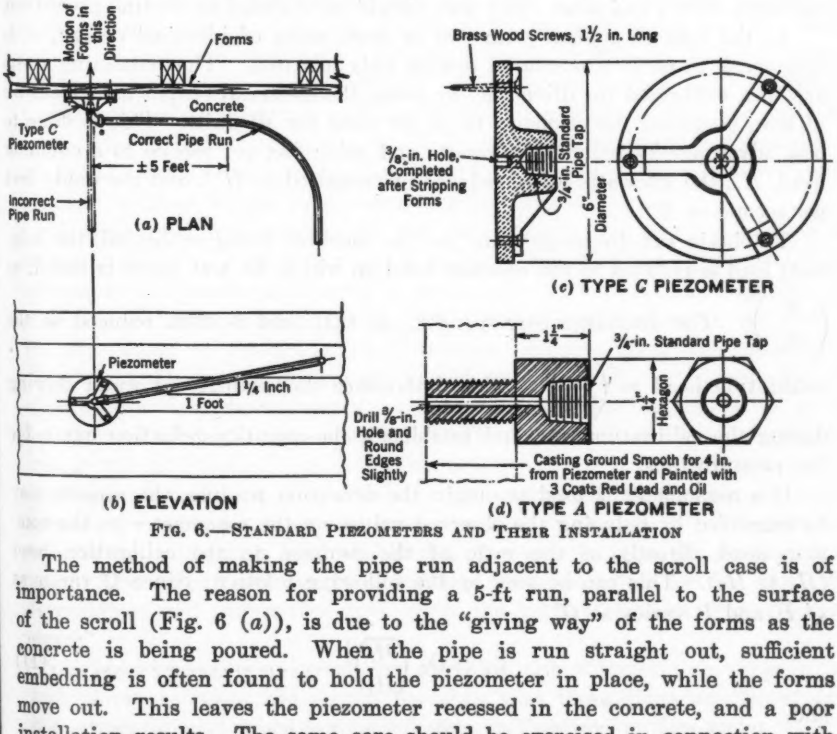


FIG. 6.—STANDARD PIEZOMETERS AND THEIR INSTALLATION

The method of making the pipe run adjacent to the scroll case is of importance. The reason for providing a 5-ft run, parallel to the surface of the scroll (Fig. 6 (*a*)), is due to the "giving way" of the forms as the concrete is being poured. When the pipe is run straight out, sufficient embedding is often found to hold the piezometer in place, while the forms move out. This leaves the piezometer recessed in the concrete, and a poor installation results. The same care should be exercised in connection with plate-steel cases as the pipe threads might be stripped in the piezometer nut, permitting leakage in the pressure line.

As the differential pressure taps in the scroll case are capable of indicating flow to a high degree of accuracy, the installation of piezometers and pressure lines should be given extreme care. Close attention should be given to tightness against leaks, proper location of valves, and careful alignment of pipe to obtain a positive upward slope to convenient valves for the elimina-

tion of air. The piezometers should have ample clearance from any projection or depression within the walls of the scroll case. A smooth surface, capable of being maintained in that condition, should surround the piezometer.

Plant Calibration.—Pressure taps in the scroll case are of considerable value during the turbine efficiency test. They provide a convenient and accurate means of comparing the measured flow through the machine with the power output, the distance traveled by the piston of the servo-motor, and net head on the turbine. Inconsistent points are weighed as to their probable value, or are discarded as being in error.

The instruments used in measuring the differential pressure should be of the highest type available. The observer should be skilled in the reading of the true pressures and in the manipulation of the pressure lines. Static readings, before and after every test, should be included as routine procedure.

In the case of a plant with two or more units of identical design, it is customary to make a measured test on only one unit. The remaining units are then calibrated for discharge, by using the electrical output as a measure of flow, assuming the efficiency to be the same for all units. This is done to best advantage by reducing kilowatts and cubic feet per second to a common head, H_c, the kilowatts being reduced or increased as $H^{1.5}$, and the cubic feet per second as $H^{0.5}$.

To obtain the discharge value for the machine being calibrated, the kilowatt load is reduced to the common head on which the test curve is based, as $\left(\frac{H_c}{H_t}\right)^{\frac{3}{2}}$. The discharge corresponding to that load is then reduced to the calibration head as $\left(\frac{H_c}{H_t}\right)^{\frac{1}{2}}$. This determines the quantity of water flowing during the calibration run, and establishes the quantity-deflection curve for the pressure taps.

If a manometer is used to obtain the deflection reading, the process may be simplified by reducing the observed values on the manometer to the common head, directly as the ratio of the common to the calibration head (H_c to H_t). This can be seen by the following relation: Since Q varies as $\sqrt{H}$ and P varies as Q^2,

$$Q_c = Q_t \left(\frac{H_c}{H_t}\right)^{\frac{1}{2}} \quad \text{.........................(11)}$$

and,

$$P_c = \frac{H_c}{H_t} P_t \quad \text{.........................(12)}$$

It has not been proved that machines of like design and installation have like efficiency. The kilowatt output of a unit is the result (not the cause), of a quantity of water flowing through the turbine. It is important, therefore, that the factors directly influencing the discharge through the runner be taken into consideration.

The principal factors are: (1) The net pressure head on the turbine; (2) the turbine gate-opening, in inches; (3) the diameter of the entrance tips of the runner blades; (4) the discharge area of the runner blades; (5) the speed of the runner; and (6) the degree of air venting into the draft-tube.

The net pressure head on the turbine is determined by net-head piezometers in the lower reaches of the penstock, and by suitable water-level gauges in the tail-race.

The gate-opening is calibrated for every tenth graduation on the governor dial. Three points between each pair of guide-vanes—one at the top, one in the center, and one in the bottom—are calipered for all gates. These values are averaged for the gate-opening curve.

The gate-opening readings are transferred to the outside of the machine by making simultaneous readings on a graduated scale clamped to the piston trunk of the servo-motor that operates the gates. Thus, the gate-opening is translated into the distance that the piston of the servo-motor travels. Consistent readings are obtained by setting the scale at zero on the servo-motor piston with a known oil pressure in the governor system. This pressure should be used at all times and for all machines when setting the scale at zero. The motion of the gates should be in the same direction for making all tests. These precautions are necessary to eliminate the effect of "gate-squeeze" and lost motion in the operating mechanism.

A typical calibration curve for the gate-opening, piston-travel relation is shown in Fig. 7. The governor-dial curve in this diagram is for operating reference and has no significance in connection with testing or calibration.

Factors 3 and 4 relate to the physical dimension of the runner and should be investigated to determine the percentage variation from the tested unit. The discharge area of the runner should be calibrated by triangulation at close points, and the developed sections measured by planimeter so as to determine the outflow areas accurately. The usual method of calibrating the area by taking three or four widths at optional points is not sufficient.

Factors 3 and 5 produce whirls in the water that fill the annular space between the guide-vanes and the runner. These are usually of no importance because the calibration head should not vary greatly from the common test head. The output and discharge relation tends to depart too far from the three-halves and square-root laws when the runner is more than 5% "off speed."

As most hydraulic units are now provided with automatic air vents in the turbine cover-plates, care should be taken that these vents are closed and the automatic mechanism is removed when calibrating the flow meter.

The advantages of making accurate readings of the piston travel to be used as a parameter for plotting the turbine discharge is shown in the flow-travel curve in Fig. 7. The original calibration of the differential pressure taps was made by using the time-pressure method of water measurement.

The subsequent test points were made by manometer readings, and these readings were converted to cubic feet per scond by the equation,

$$Q = 1\,448\,(Hg)^{0.495} \ldots\ldots\ldots\ldots\ldots\ldots (13)$$

in which, (Hg) denotes inches of mercury. Table 1 shows the agreement of discharge measurement as determined by the manometer and as read on the rate-of-flow scale of a standard Venturi tube register after three years of service.

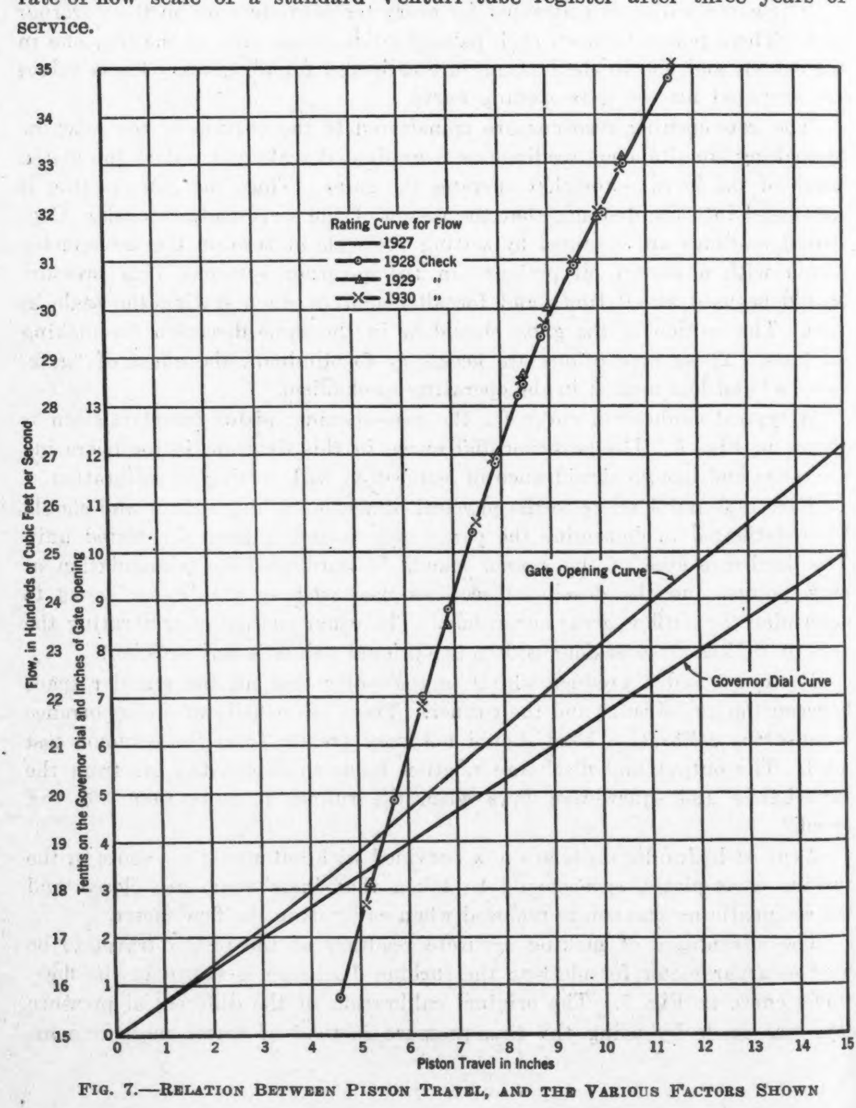


FIG. 7.—RELATION BETWEEN PISTON TRAVEL, AND THE VARIOUS FACTORS SHOWN

TABLE 1.—COMPARISON OF READING BY MANOMETER AND FLOW REGISTER, IN CUBIC FEET PER SECOND

Run	By manometer	By register	Run	By manometer	By register
1	880	878	7	3 100	3 085
2	1 340	1 330	8	3 300	3 300
3	1 770	1 764	9	3 525	3 523
4	2 180	2 172	10	3 210	3 210
5	2 545	2 528	11	2 980	2 968
6	2 850	2 840	12	2 690	2 681

Table 1 lists the actual test values from which the points for the 1930 check test, indicated in Fig. 7, were computed. The installation of equipment is identical to that shown in Fig. 5.

Plant Performance.—The performance of a typical installation of pressure taps in a plate-steel scroll case, is shown in Fig. 8. The piezometer numbers, 1 and 4, 2 and 4, and 3 and 4, refer to the corresponding numbers in Fig. 5. The curves represent test values as determined by using the salt-velocity method of water measurement.

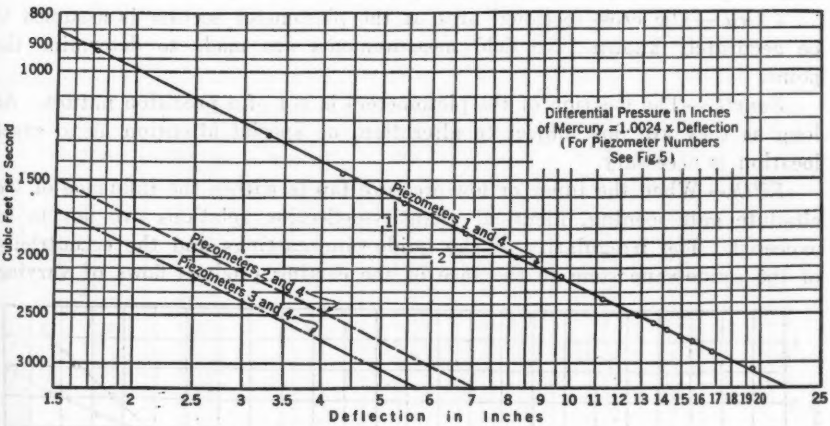


FIG. 8.—PERFORMANCE OF DIFFERENTIAL PRESSURE TAPS IN SCROLL CASE

By inspection, the exponent, n, in Equation (9) is 0.500 for all three combinations of taps. There is no evidence of a change of flow around the speed-ring stay-vane; nor is there a change of the quantity-deflection relation due to the centrifugal force of the water flowing around the short radius of the speed-ring casting. Piezometer Nos. 2 and 4 give deflections in the ideal range for standard Venturi tube registers.

Three similar machines in the same plant were calibrated for turbine discharge by reference to the kilowatt-discharge values obtained during the test, as shown in Fig. 8. The following deflections, in inches of mercury,

for Taps Nos. 1 and 4 (see Fig. 5) were found, corresponding to a discharge of 3 000 cu ft per sec:

Turbine Unit No.	Deflection, in inches of mercury.	Percentage variation from the average.
1	21.3	+ 10
2	18.2	— 6
3	19.3	— 1
4	19.0	— 2

The percentages of variation of the four units was found by averaging the deflections. These values are greater than those found for piezometers corresponding to Taps Nos. 2 and 4, and 3 and 4. The predicted and observed deflection for the more conservative designs seldom differ more than 5 per cent.

The reasons for a difference in the pressures found in similar installations are as follows, in order of their importance:

First.—The pressure taps are designed assuming that the water is distributed equally around the unit. This condition does not exist.

Second.—The efficiency is assumed to be the same for like units. Actual data on plant tests, confirming this point, are lacking.

Third.—The cross-sectional area at the piezometer section is assumed to be accurately known. No field measurements are made to determine the point.

Fourth.—The location of the piezometers is not of a precision nature. As long as they are not subject to alteration, no special attention as to exact location is necessary.

Fifth.—When the inner or low-pressure tap is within the influence of the absolute gate-opening, identical quantity-deflection relations are not to be expected. The irregularity of the guide-vane castings and the eccentricity of the guide-vane stem to the face of the casting produce flows of varying

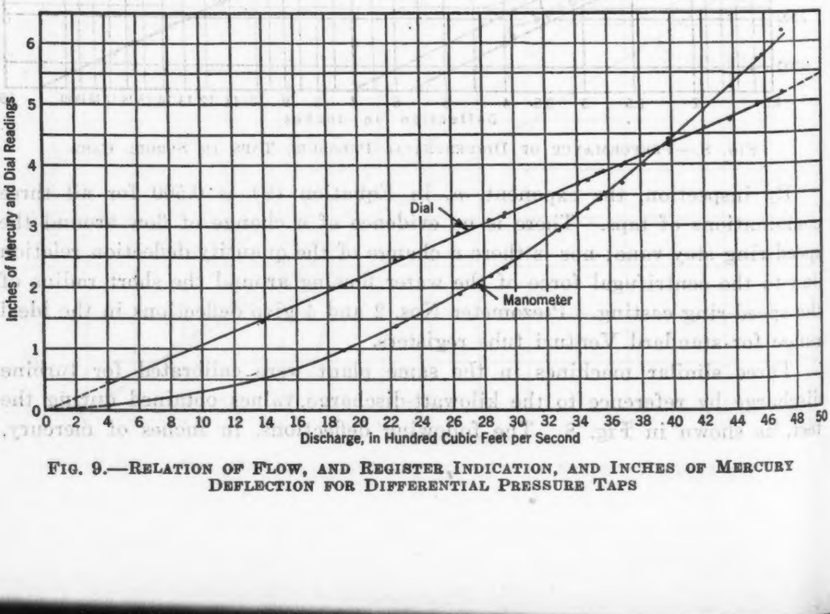


FIG. 9.—RELATION OF FLOW, AND REGISTER INDICATION, AND INCHES OF MERCURY DEFLECTION FOR DIFFERENTIAL PRESSURE TAPS

quantities. This fact becomes of first importance in installations using Piezometers Nos. 1 and 4 (Fig. 5).

An interesting exhibit of plant results is shown in Fig. 9. These curves are from plant-test data made by the salt-velocity method of water measurement, and are for the plant shown in Fig. 4. The piezometer installation is as shown in Fig. 2, Taps Nos. 1 and 2. The manometer readings, in inches of mercury, compare with the laboratory results, in inches of water, as shown in Fig. 3. The comparative agreement between the laboratory and plant curves is within 2%, the laboratory values being consistently lower.

By plotting the actual test points as given in Table 2, the fine agreement between the measured discharge, manometer, and dummy dial reading of the meter register, can be studied. The readings were obtained on instruments installed as shown in Fig. 5. The meter used is a standard register, designed to operate with a Venturi tube. The flow scale, or dummy dial, is graduated, in inches of the arc traveled by the pointer, and is an arbitrary value used for reference purposes only. By installing the meter register before making the plant test, the values for final calibration are read directly on the scale during the efficiency test. The engraving of a new flow scale is a simple matter, necessitating only the shifting of the instrument zero and re-spacing of the graduations to read in cubic feet per second.

TABLE 2.—PLANT EFFICIENCY TEST DATA

Run	Head, H_t, in feet	Measured discharge, in cubic feet per second	Differential pressure, P, = inches of mercury	Readings, register dummy dial	Horse-power	Inches of piston travel	Inches of gate-opening	Readings on governor dial, in tenths
1....	91.99	1 381	0.51	1.46	10 194	4.18	3.15	3.0
2....	91.97	1 907	0.97	2.04	15 526	5.78	4.45	4.0
3....	91.84	2 639	1.81	2.87	23 933	7.56	5.95	5.0
4....	91.81	2 930	2.33	3.17	26 515	8.28	6.60	5.5
5....	91.79	3 269	2.92	3.53	30 649	9.25	7.45	6.0
6....	91.81	3 527	3.42	3.84	33 233	9.93	8.05	6.5
7....	92.01	3 806	4.00	4.15	35 430	10.93	8.90	7.0
8....	91.82	3 984	4.36	4.42	37 004	11.65	9.50	7.5
9....	91.65	4 207	4.87	4.59	38 312	12.50	10.20	8.0
10....	91.56	4 377	5.26	4.74	39 000	13.25	10.80	8.5
11....	91.33	4 555	5.70	4.96	39 586	14.12	11.50	9.0
12....	91.10	4 719	6.15	5.16	39 902	15.06	12.25	10.0
13....	92.14	2 236	1.36	2.43	19 457	6.62	5.15	4.5
14....	92.09	2 935	2.35	3.20	26 906	8.37	6.65	5.5
15....	92.01	3 291	2.96	3.58	30 986	9.28	7.45	6.0
16....	91.97	3 402	3.21	3.71	32 647	9.62	7.75	6.2
17....	91.98	3 465	3.28	3.76	32 843	9.72	7.85	6.4
18....	91.92	3 544	3.43	3.86	33 773	9.97	8.06	6.5
19....	91.92	3 534	3.44	3.86	33 611	9.97	8.06	6.5
20....	91.88	3 567	3.55	3.89	34 196	10.09	8.15	6.6
21....	92.18	3 681	3.71	3.99	34 657	10.40	8.40	6.7
22....	92.13	3 701	3.77	4.01	34 854	10.53	8.55	6.8
23....	92.04	3 825	4.00	4.15	35 622	10.97	8.91	7.0
24....	92.09	402	0.05	0.46	1 676	1.25	0.90	1.0
25....	92.09	....	0.00	0.00	—228	0.00	0.00	0.0

It is significant to note that the quantity-deflection relation for the differential pressure taps, follows those of the Venturi tube sufficiently close to enable the use of standard Venturi tube equipment.

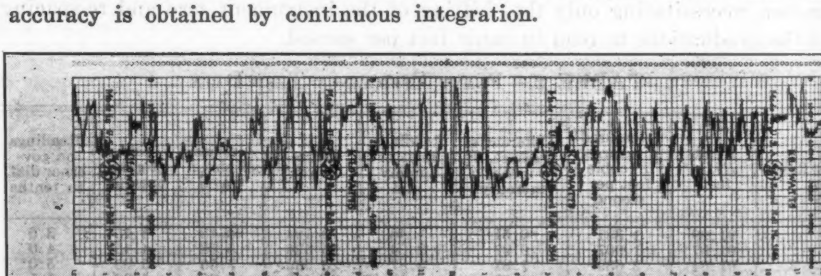
The plotting of the curves in Fig. 9 clearly indicates that a deflection of 6 to 8 in. of mercury is sufficient for all practical purposes. A greater deflection must carry the same error in percentage, and, therefore, can have no

advantage over the lower deflection for absolute values. Furthermore, as the rating of the pressure taps is dependent upon the measured flow, a degree of accuracy greater than that of the measuring means is of no value.

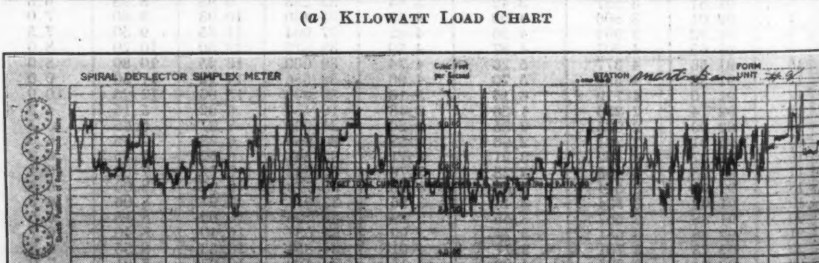
The height of the diagrams obtained in the time-pressure and salt-velocity methods of water measurement do not ordinarily exceed 3 or 4 in.; and a second dimension, time, is introduced in the computation for flow. In the case of the manometer and dummy dial reading, only one dimension is involved. With a deflection of 6 to 8 in. of mercury for the differential taps, the advantages are obvious.

In the case of a calibration test for similar machines where the electrical output is used to determine the turbine discharge, the same relation exists. A high-grade portable watt-meter will ordinarily show a scale deflection of 2 to 3 in. The water-meter register will have a corresponding scale deflection of from 5 to 8 in.

The water-meter register is capable of continuous integration with a prime moving force in the same proportion as the deflection. This method of rating can be used in connection with the measurement of water by current meter and by rotating standards for electrical output, if the engineer believes greater accuracy is obtained by continuous integration.



(*a*) KILOWATT LOAD CHART



(*b*) WATER INPUT RATE CHART FOR HYDRAULIC TURBINE

FIG. 10.—TYPICAL KILOWATT-LOAD AND WATER-INPUT CHARTS

Fig. 10 (*a*) is a sample record chart showing the flow characteristic obtained for a turbine equipped with a water register connected to differential pressure taps. The corresponding record chart (Fig. 10 (*b*)) of electrical output is included for purposes of comparison.

Plant Operation.—The operating advantages to be had by reference to water-measuring means are not fully realized. The general method in use at present to arrive at the daily operating efficiency is by reference to the kilowatt-hours per hour reading. This value is converted to cubic feet per second by assuming the unit operating at one load, and at constant head. Unless the plant has been on base load for a long time, such methods give results that are not in agreement with the facts.

The following example illustrates the errors possible:

Total kilowatt-hour per hour reading....	=	24 500
Over-all efficiency at 24 500 kw..........	=	89.5%
Apparent operating efficiency...........	=	89.5%

The load was actually carried as follows:

45 min at 29 000 kw....................	=	82.0%
15 min at 11 200 kw....................	=	76.0%

Then, the weighted average efficiency is:

82 × 29 000.............................	=	2 380 000
76 × 11 200.............................	=	850 000
Σ kw......40 200		3 230 000 = Σ products

$$\text{Efficiency} = \frac{3\,230\,000}{40\,000} = 80 \text{ per cent.}$$

Therefore, the error in the efficiency value by using the kilowatt-hour per hour reading is $89.5 - 80.0 = 9.5$ per cent. The absolute error in quantity of water flowing through the unit is $\frac{89.5}{80.0} = 1.12$, or 12 per cent. The change in tail-race elevation between hourly readings can frequently account for errors of 2 or 3% in efficiency and flow values.

Concrete proof of the inability of operating engineers to secure the best results by the use of an efficiency curve only is shown by actual plant records. Increases in over-all plant efficiency amounting to 4% in generation have been effected by the use of water registers.

The water power generated in the United States accounts for about 6% of the total generation. As water power can be brought on the line within 1 min to 3 min, with maximum efficiency, the importance of hydro-electric plants as peak-load power can be seen. As the ratio of hydro-electric to steam generation decreases, the necessity of water-recording will increase.

On rivers being developed for power throughout their length, the formation of slack-water pools drowns out the stream-gauging stations. Unless plant-recording devices are provided, the very necessary stream-flow records will cease.

In the case of joint use of a stream by two or more companies, the quantity of water flowing must be known because charges are made for regulated flows. Any method, subject to possible errors as shown to exist when positive recording devices are not provided, might lead to expensive controversies over water rights.

Conclusions

The following summary of conclusions is offered for discussion, based on the evidence presented in the paper.

(1) Laboratory tests on models of scroll cases indicate that the pressure differences on opposite walls of the scroll case can be used to indicate the rate of flow through the turbine.

(2) These pressures were found to follow definite laws of motion, and a flow relation could be established involving fundamentally sound principles.

(3) Laboratory investigations show that a change in head on the plant will not alter the quantity-deflection relation.

(4) The differential pressures in the scroll case are related to the flow through the turbine with characteristics of the Venturi tube. These characteristics are sufficiently close to enable the use of standard Venturi-tube instruments.

(5) The exponent, n, in Equation (10) ($Q = k P^n$) is substantially 0.500. The information being compiled on various plant tests indicates that a true square law relation will agree more uniformly with plant calibrations than the Venturi-tube exponent.

(6) There is no definite information indicating a change in quantity-deflection relation at low flows, as is manifested in the Venturi tube with throat velocities less than 5 ft per sec.

(7) Plant records indicate that the differential pressure taps in the scroll case will give consistent values over a period of years.

(8) The coefficient, C, in Equation (3) was found to cover a wide range, but in agreement with the fundamental laws used as a basis for design.

(9) By application of the laws of similitude, the experimental coefficient, C, can be used to predict the quantity-deflection relation within 5% of the observed value.

(10) Plant recording and operation can be brought to a high degree of efficiency by the use of water-recording devices.

(11) Experience with the scroll-case taps to date, indicate that they are not satisfactory for determining the rate of flow through a turbine without first being calibrated by other means of water measurement.

Acknowledgment

The writer wishes to express his appreciation to the Alabama Power Company, W. S. Barstow and Company, and the Simplex Valve and Meter Company for their co-operation in supplying a large part of the data presented in the paper; and to Mr. Kennedy for his valuable aid in the conception and development of the flow meter.

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PAPERS

THREE-SPAN CONTINUOUS-TRUSS RAILROAD BRIDGE, CINCINNATI, OHIO

By Wilson T. Ballard[1], M. Am. Soc. C. E.

Synopsis

A description of the newest bridge of the Chesapeake and Ohio Railway Company across the Ohio River at Cincinnati, Ohio, is given in this paper. The recorded history of this important crossing begins with a paper[2] by William H. Burr, M. Am. Soc. C. E., entitled "The River Spans of the Cincinnati and Covington Elevated Railway, Transfer and Bridge Company." The purpose of the present paper is to provide the next chapter of this history. The bridge built from 1927 to 1929 to replace the old one (which was completed in 1888, and strengthened during 1916) is a double-track steel structure, continuous over three spans.

Features of the new bridge that are of special interest are: (1) The trusses (which are the longest of their kind constructed to date); (2) the location of a fixed bearing at the end of the bridge, necessitating an exceedingly heavy anchorage pier of unusual design; (3) a river pier comprising an old, stone masonry part on a timber caisson joined with a new concrete part on a concrete caisson; (4) erection without inconvenience to railroad traffic on the old bridge, or to navigation of the river; and (5) the use of silicon alloy steel for the purpose of reducing dead load.

The Old Bridge

The old double-track bridge of the Covington and Cincinnati Elevated Railroad and Transfer and Bridge Company, constructed in 1886 to 1888, provided the Chesapeake and Ohio Railway Company and the Louisville and Nashville Railroad Company an entrance to Cincinnati, Ohio, from the east and south. The railroad tracks were between the trusses. A 10.5-ft wide,

Note.—Presented at the meeting of the Construction Division, New York, N. Y., January 22, 1931. Discussion on this paper will be closed in August, 1933, *Proceedings*.

[1] Vice-Pres., The J. E. Greiner Co., Baltimore, Md.

[2] *Transactions*, Am. Soc. C. E., Vol. XXIII (August, 1890) p. 47.

vehicular roadway and a sidewalk, 5.75 ft wide, were supported on brackets outside the trusses on each side of the bridge. The trusses were designed for a live load on each track consisting of two Consolidation engines weighing 119 000 lb each (exclusive of tender), followed by a uniform train load of 2 500 lb per ft; and a highway live load on each wagon way and sidewalk, of 60 lb per sq ft, which is equivalent to 960 lb per lin ft per truss.[3]

The floor system and primary truss members, in addition to the foregoing loads, were designed for a load of 15 tons uniformly distributed in 10 ft, followed, and preceded by, a uniform load of 80 lb per sq ft, on each vehicular roadway and sidewalk. This loading was applied alternately on each side of the bridge. The floor system and lateral and transverse systems of bracing were wrought iron. All main truss members were steel.

An inspection of the bridge made during June, 1916, disclosed the fact that the bridge was carrying locomotives approximately 55% heavier, and train loads, 100% heavier, than those for which the structure was designed. All operation of these loads over it was limited to 15 miles per hour. Subsequently, the bridge was strengthened in some of its members, and it continued to carry the foregoing loads safely for a period of thirteen years until, in March 1929, the new bridge was put into service.

The first bridge was maintained in service as a railroad bridge for a period of about forty-one years. It has now been converted into a highway bridge, which should continue in service indefinitely, or for as long a period as its owners take proper care of it.

Description of the Project

During 1926 the Chesapeake and Ohio Railway Company began extensive improvements in Covington, Ky., and in Cincinnati. This work included: (*a*) Elimination of several grade crossings in Covington, necessitating a material rise in grade on the approach to the Ohio River Bridge; (*b*) construction of new passenger platforms and train-sheds in Covington; (*c*) revision of viaducts and trackage in Cincinnati; and (*d*) replacement of the old bridge by a new double-track bridge designed for Cooper's E-70 live load. The construction of the new bridge across the Ohio River was justifiably included in the program of improvements, because it would permit the Railway Company to operate its heaviest equipment between Covington and Cincinnati, with resulting improved efficiency and economy in handling cross-river traffic, instead of being limited, in hauling trains across the river, to the use of power approximately equivalent to Cooper's E-40 loading.

General Conditions and Selection of Design

The old bridge (see Fig. 1) consists of three, simple, through-truss spans, resting on stone masonry piers. The river piers rest on cribs of 12 by 12-in. pine timbers with interstices filled with concrete. These cribs, which are approximately 35 ft high, were constructed on caissons built of pine timber with oak cutting-edges and working chambers filled with concrete. Explora-

[3] *Transactions*, Am. Soc. C. E., Vol. XXIII (August, 1890), p. 62.

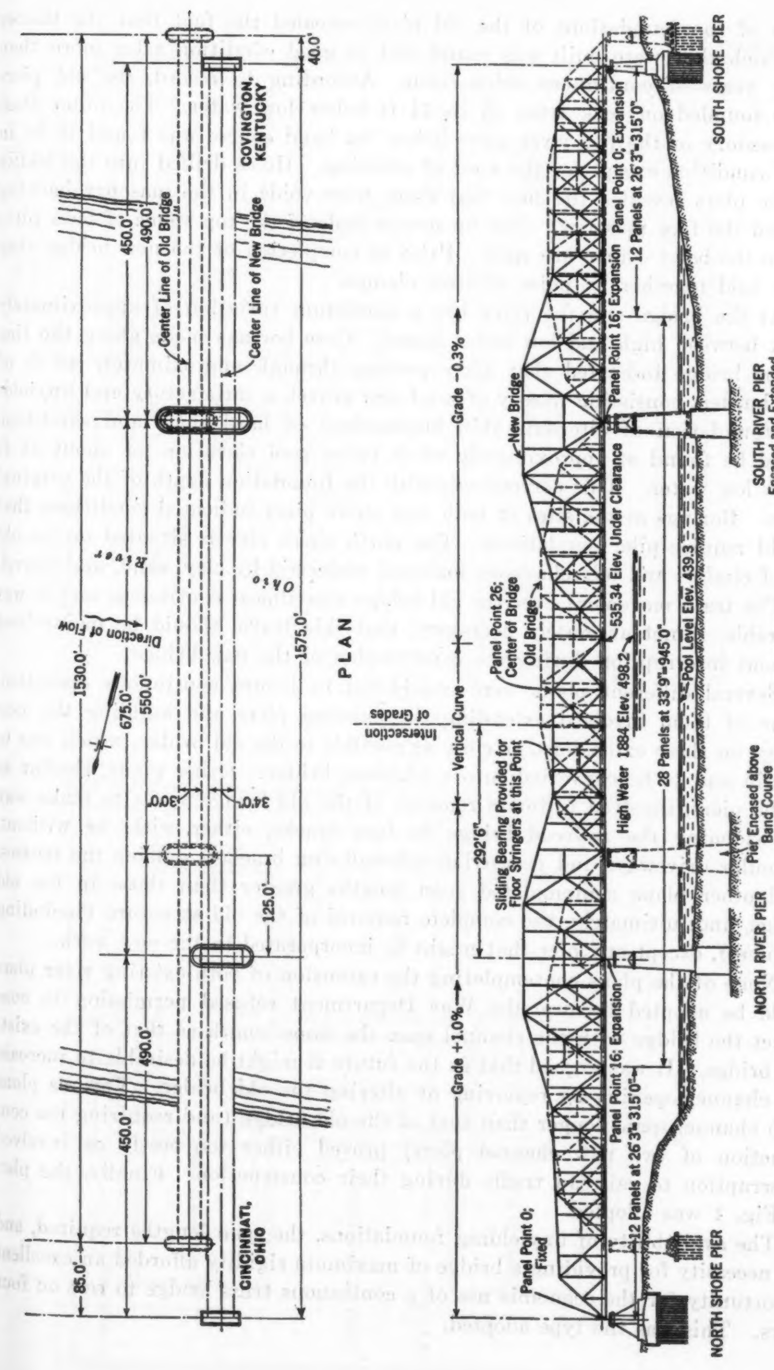


FIG. 1.—PLAN AND ELEVATION OF BRIDGE

tions of the foundations of the old piers revealed the fact that the timber of which they were built was sound and in good condition after more than forty years of continuous submersion. According to record, the old piers were founded on rock from 53 to 54 ft below low water. The outer shell of masonry of the two river piers below the band course was found to be in good condition except for the need of pointing. Holes drilled into the bodies of the piers revealed the fact that there were voids in the masonry backing behind the face masonry. The up-stream ends of the top stems of both piers above the band course are split. Prior to completion of the new bridge they were held together by pairs of steel clamps.

At the bridge site the river has a maximum variation of approximately 70 ft between high and low-water stages. Core borings made along the line of the bridge indicated that after passing through approximately 40 ft of over-burden, consisting mostly of sand and gravel, a satisfactory and unyielding foundation of alternate thin laminations of hard shale and sandstone would be found at approximately 66 ft below pool elevation, or about 54 ft below low water. This corresponds with the foundation depth of the original piers. Borings at the sites of both new shore piers indicated conditions that would require pile foundations. The north shore pier is situated on an old fill of cinders and miscellaneous material underlaid by clay, sand, and gravel.

The train movement over the old bridge was almost continuous and it was desirable, if not absolutely necessary, that this traffic should be maintained without interruption during the construction of the new bridge.

Several different plans were considered to insure continuous operation. Some of them involved extending the existing piers and building the new bridge on these extensions, as close as possible to the old bridge, which was to remain and to be converted into a highway bridge. Other plans, similar as to the piers, involved ultimate removal of the old bridge spans to make way for widening the railroad bridge to four tracks, either with, or without, vehicular driveways and pedestrian sidewalks on brackets outside the trusses. Still other plans contemplated span lengths greater than those in the old bridge, and, ultimately, the complete removal of the old structure (including all piers), except any pier that might be incorporated in the new work.

None of the plans contemplating the extension of both existing river piers could be adopted because the War Department refused permission to construct the bridge with the channel span the same length as that of the existing bridge. It was argued that in the future it might be desirable to increase the channel opening by removing or altering the old bridge. Various plans with channel spans longer than that of the old bridge (and requiring the construction of two new channel piers) proved either too costly or involved interruption to railroad traffic during their construction. Finally, the plan in Fig. 1 was adopted.

The availability of unyielding foundations, the span lengths required, and the necessity for providing a bridge of maximum rigidity afforded an excellent opportunity for the economic use of a continuous truss bridge to rest on four piers. This was the type adopted.

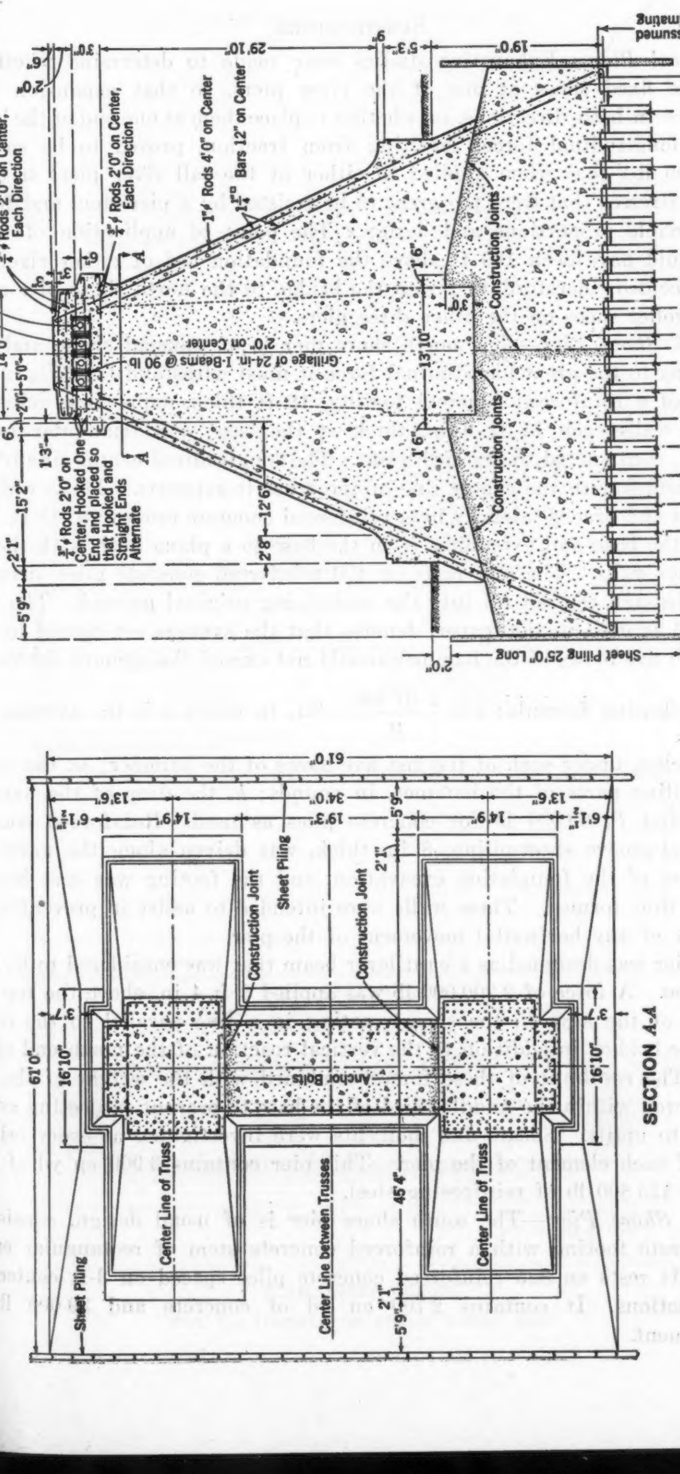


FIG. 2.—DETAILS OF NORTH SHORE PIER

Substructure

General Plan.—Exhaustive studies were made to determine whether to place the fixed shoes at one of the river piers, so that expansion would take place in both directions, or whether to place them at one end of the bridge.

The longitudinal force, resulting from traction, proved to be so great (2 200 000 lb) that when applied to either of the tall river piers it caused bending stresses that were too great to be resisted by a pier stem and footing of reasonable proportions and design. The point of application of such a force would have been 160 ft above the foundation bed of either river pier. It was decided, therefore, to fasten the bridge to the north shore pier and to provide roller shoes on the other three piers.

North Shore Pier.—The north shore pier is of unusual shape and large size, owing to the great longitudinal force it must withstand (see Fig. 2). It consists of a reinforced concrete footing, 18 ft thick, supporting reinforced concrete walls each 14 ft 9 in. thick at the base, and triangular in side elevation, constructed under each shoe. The longitudinal center line of each wall coincides with the center line of the truss it supports. These walls are connected and braced laterally by a reinforced concrete cross-wall, 13 ft 10 in. thick at the base and extending from the base to a plane flush with the tops of the side walls. The pier rests on 400 reinforced concrete piles, driven to refusal through the old fill into the underlying original ground. The term, "refusal," as used in this paper, denotes that the average set caused by each of the last five blows of the hammer should not exceed the amount determined by the following formula: $s = \frac{1.07\, wh}{P} - 0.1$, in which s is the average final set, in inches, under each of the last five blows of the hammer; w, the weight of the falling parts of the hammer, in pounds; h, the drop of the hammer, in feet; and P, 60 000 lb for concrete piles as used. Reinforced concrete tongue-and-groove sheet-piling, 8 in. thick, was driven along the north and south sides of the foundation excavation, and the footing was cast between the walls thus formed. These walls were intended to assist in preventing the possibility of any horizontal movement of the pier.

The pier was designed as a cantilever beam that was considered to be fixed at the base. A force of 2 200 000 lb was applied 3 ft 4 in. above the top (the elevation of the keys in the shoes), acting in a line parallel to the center line of the bridge, in addition to the vertical reaction of the north end of the bridge. The resultant of these forces combined with the weight of the pier was kept well within the middle-third of the base to prevent subjecting any of the piles to uplift. Shears and moments were investigated at every critical section of each element of the pier. This pier contains 3 900 cu yd of concrete and 125 800 lb of reinforcing steel.

South Shore Pier.—The south shore pier is of usual design, consisting of a concrete footing with a reinforced concrete stem of rectangular cross-section. It rests on 325 reinforced concrete piles spaced on 3-ft centers in both directions. It contains 2 700 cu yd of concrete and 39 000 lb of reinforcement.

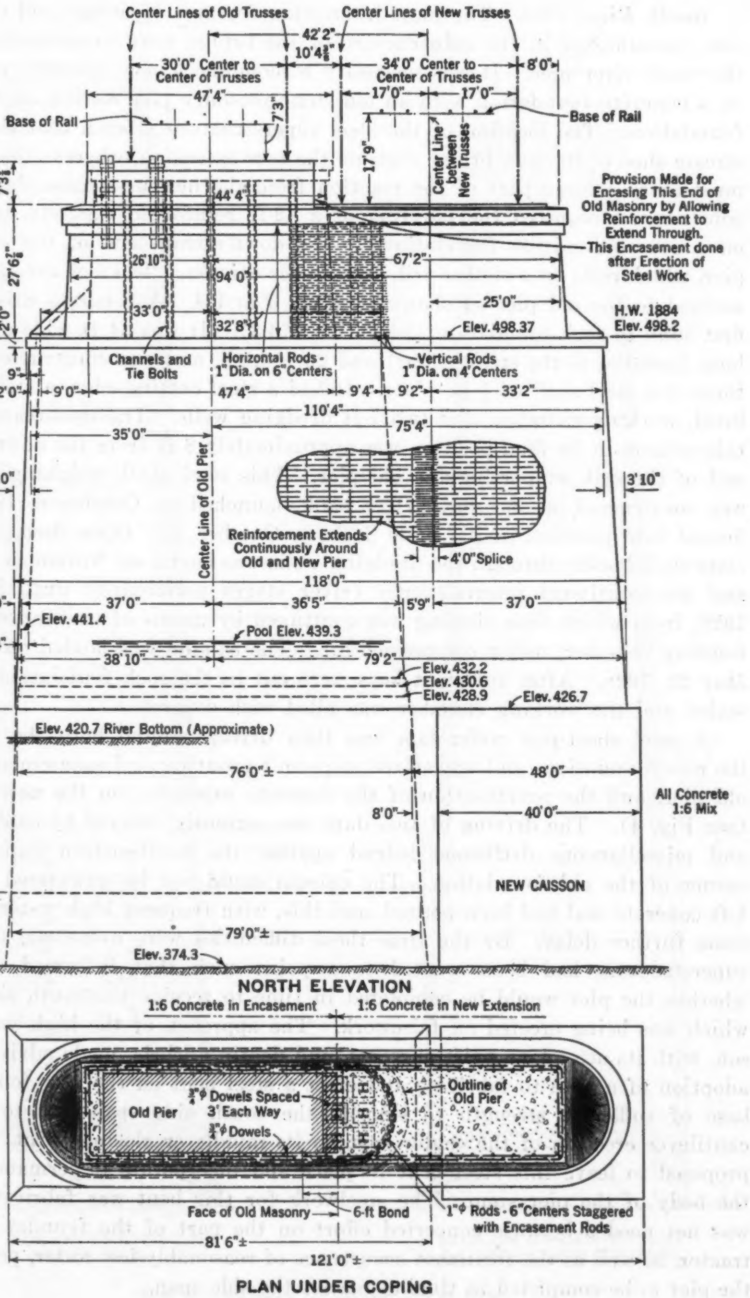


FIG. 3.—DETAILS OF SOUTH SHORE PIER

South River Pier.—The most interesting features of design and construction encountered in the substructure of the bridge were in connection with the south river pier. It was necessary to combine a new concrete extension on a concrete foundation with an old stone masonry pier resting on a timber foundation. The location of the steel superstructure is such that the down-stream shoe of the new bridge rests on the new extension, whereas the old pier must carry a large part of the reaction from the new up-stream shoe. This construction required the building of a 42-ft reinforced concrete extension, on a concrete caisson foundation, to the down-stream end of the old stone pier, which rests on a timber crib and timber caisson. The new extension was secured to the old pier as shown in Figs. 3 and 4. A concrete caisson was first sunk to rock foundation (Elevation 374.3). It was 34 ft wide and 40 ft long (parallel to the stream flow) and 25 ft high, and was constructed in the form of a steel shell of $\frac{1}{4}$-in. plate. It had a steel cutting-edge, a steel, plate-lined, working chamber, and two 7-ft dredging wells. The up-stream end of this caisson in its final position was approximately 8 ft from the down-stream end of the crib supporting the old pier. This steel shell, weighing 58 tons, was constructed on the north river bank, launched on October 6, 1927, and floated into position in its guide dock on October 17. Open dredging by a clam-shell bucket through the dredging wells was begun on November 1, 1927, and was continued intermittently (river stages permitting) until May 12, 1928, from which time sinking was continued by means of excavation in the working chamber, under compressed air. The caisson was landed on rock on May 21, 1928. After sufficient keys were cut in the rock foundation, it was sealed and the working chamber was filled with concrete.

A steel sheet-pile coffer-dam was then driven around both the old and the new foundations and unwatered to permit grouting and encasement of the old pier, and the construction of the concrete extension on the new caisson (see Fig. 4). The driving of this dam was seriously delayed by sunken logs and miscellaneous driftwood lodged against the northeastern (up-stream) corner of the old foundation. The caisson could not be unwatered until a 4-ft concrete seal had been poured, and this, with frequent high water, caused some further delay. By the time these difficulties were overcome, the steel superstructure had been erected to a point such that it semed doubtful whether the pier would be completed in time to receive the south side span which was being erected on falsework. The approach of the high-water season, with its attendant swift currents, and floating débris, made advisable the adoption of a plan to fabricate and erect a steel bent on the new foundation base of sufficient strength to support the south side span and to permit cantilever erection of the center span to its middle or closing point. It was proposed to leave this steel bent in place and incorporate it permanently in the body of the pier stem. The steelwork for this bent was fabricated, but was not needed because concerted effort on the part of the foundation contractor, as well as the fortunate occurrence of reasonably low water, permitted the pier to be completed in time to receive the side span.

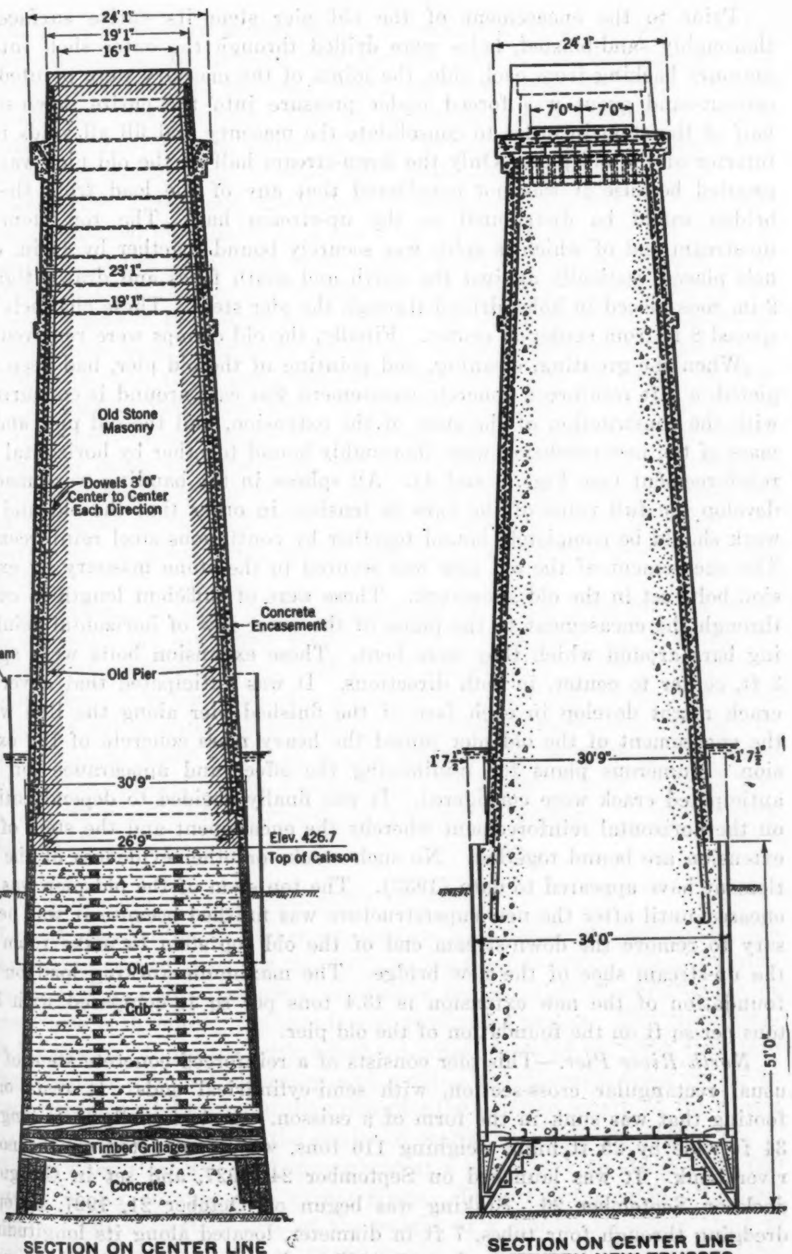


FIG. 4.—DETAILS OF SOUTH RIVER PIER

Prior to the encasement of the old pier stem its entire surface was thoroughly sand-blasted, holes were drilled through the outer shell into the masonry backing from each side, the joints of the masonry were pointed, and cement-sand grout was forced under pressure into the entire down-stream half of the stem in order to consolidate the masonry and fill all voids in the interior of the old pier. Only the down-stream half of the old pier was thus grouted because it was not considered that any of the load from the new bridge would be distributed to the up-stream half. The top stem, the up-stream end of which is split, was securely bound together by 15-in. channels placed vertically against the north and south faces and drawn tight by 2-in. rods placed in holes drilled through the pier stem. These channels were spaced 8 ft from center to center. Finally, the old clamps were removed.

When the grouting, cleaning, and pointing of the old pier, had been completed, a 2-ft reinforced concrete encasement was cast around it concurrently with the construction of the stem of the extension, and the old pier and the mass of the new extension were thoroughly bound together by horizontal steel reinforcement (see Figs. 3 and 4). All splices in the banding were made to develop the full value of the bars in tension, in order that the old and new work should be completely bound together by continuous steel reinforcement. The encasement of the old pier was secured to the stone masonry by expansion bolts set in the old stonework. These were of sufficient length to extend through the encasement to the plane of the outer row of horizontal reinforcing bars around which they were bent. These expansion bolts were spaced 3 ft, center to center, in both directions. It was anticipated that a vertical crack might develop in each face of the finished pier along the line where the encasement of the old pier joined the heavy mass concrete of the extension. Numerous plans for minimizing the effect and appearance of this anticipated crack were considered. It was finally decided to depend entirely on the horizontal reinforcement whereby the encasement and the stem of the extension are bound together. No such cracks or other evidences of the settlement have appeared to date (1933). The top stem of the old pier was not encased until after the new superstructure was finished because it was necessary to remove the down-stream end of the old top stem to make room for the up-stream shoe of the new bridge. The maximum working load on the foundation of the new extension is 13.4 tons per sq ft compared with 14.8 tons per sq ft on the foundation of the old pier.

North River Pier.—This pier consists of a reinforced concrete stem of the usual rectangular cross-section, with semi-cylindrical ends. It rests on a footing that was sunk in the form of a caisson. A steel shell, 84 ft long by 34 ft wide by 25 ft high, weighing 116 tons, was constructed on the north river bank. It was launched on September 24, 1927, and set in the guide dock on September 26. Sinking was begun on October 21, 1927, by open dredging through four tubes, 7 ft in diameter, located along its longitudinal center line. Between November 17, 1927, and January 10, 1928, little or no progress was made because of high-water stages. From about the middle of January, 1928, the work progressed almost continuously until May 28, 1928,

at which time locks were erected on the tubes and sinking was continued by the pneumatic method. At the time air was applied the cutting-edge was approximately 7 ft above the final rock foundation, upon which the caisson was landed on June 9, 1928. Due to high-river stages (several of which necessitated temporary suspension of work in the caisson), the caisson was not sealed nor the working chamber filled until July, 1928.

Construction Plant and Methods.—The contractor's plant for the substructure consisted essentially of a tower on the north bank of the river, a mixing plant at the base of the tower, and a narrow-gauge track for a gasoline-propelled train of concrete buggies on the vehicular roadway along the down-stream side of the old bridge. The mixing plant consisted of the usual sand and gravel bins, water tank, and a 1-yd mixer. Concrete was made and distributed to both shore piers and the north river pier by means of this plant. A floating plant was used for making and placing concrete in the south river pier, because when this pier was being poured the down-stream roadway was being removed from the old bridge to permit steel erection. In order to facilitate concreting the north river pier, the concrete tower was moved from the north bank to a location in the river near that pier. It was supported on piles driven in the river and was securely fastened to the old bridge.

All concrete was made in the proportions of 1 part cement to 6 parts aggregate, with the sand and gravel measured separately. A combination of 2.3 parts of sand with 3.7 parts of gravel was found to produce concrete of maximum density which, when mixed for not less than 1 min in the mixer with from $5\frac{2}{3}$ to 6 gal of water per sack of cement, attained an average strength at 28 days of about 2 500 lb per sq in., as indicated by cylinder tests. Three pounds of an admixture of diatomaceous silica per bag of cement was used, and this assisted materially in producing concrete of satisfactory workability.

Superstructure

General Description.—Numerous studies were made to determine the proper character and lines for the 1 575-ft, continuous truss. Consideration of appearance, as well as economy, resulted in the selection of a triangular, or Warren, type with sloping top chords and subdivided panels. The trusses are 34 ft from center to center, with top-chord cover-plates 44 in. wide, thus providing adequate railroad clearance and making the distance from center to center of trusses slightly more than one-twentieth of the length of the center span. Other dimensions and the grades of roadway, are shown in Fig. 1.

Silicon alloy steel was used throughout at a basic working stress of 24 000 lb per sq in. for all steelwork, except rivets. The resultant savings in steel weight was approximately 19% of the weight of a similar structure built of structural grade carbon steel at 18 000 lb per sq in. Movable shoes on steel rollers, 24 in. in diameter, enclosed in oil boxes, were provided on the south shore pier and on both river piers.

The silicon steel was manufactured by the open-hearth process according to specifications which required the following chemical properties:

	Ladle analysis	Check analysis
Carbon, maximum percentage	0.40	0.44
Phosphorous, maximum percentage:		
Acid	0.06	0.075
Basic	0.04	0.05
Sulfur, maximum percentage	0.05	0.063
Silicon, minimum percentage	0.20	0.18

The specified ultimate strength was 80 000 to 95 000 lb per sq in., with a minimum yield point of 45 000 lb per sq in. A review of the tests shows that the elastic limit of the material obtained was consistently from 2 000 to 4 000 lb per sq in. in excess of the minimum requirement, with silicon content ranging between 0.2 and 0.3 per cent.

Design Loads and Stresses.—Cooper's E-70 engine loading was used in computing the live load stresses in the floor system, hangers, and subdiagonals. The impact increments added to stresses thus determined were based on the formula:

$$I = S\,\frac{2\,000 - L}{1\,600 + 10\,L} \qquad (1)$$

in which, I = impact increment; S = computed maximum live load stress, in pounds per square inch; and L = loaded length of track, in feet, producing the maximum live load stress in the member. This includes the length of both tracks, loaded, for members receiving a maximum live load stress for this condition.

The trusses were designed for 90% of Class E-70 engine loading, which for the purpose of computing stresses was represented by a load of 6 300 lb per ft of truss, with a load of 95 000 lb concentrated at each of two panel points in each truss, separated by two panel lengths. In determining maximum stresses, full consideration was given to the effect of split loads using the aforementioned concentrations.

Impact increments of the live load stresses in main truss members were found by using the following formulas:

For stresses of single character:

$$I = \frac{300}{300 + L} \times \frac{\text{Live Load}}{\text{Live Load} + \text{Dead Load}} \qquad (2)$$

For reversal of stresses:

$$I = \frac{300}{300 + L} \times \left(1 + \frac{m}{2\,M}\right) \qquad (3)$$

in which, M = the maximum algebraic sum of dead and live load stresses of like character; and m = the maximum algebraic sum of dead and live load stresses of opposite character. Equation (3) was not used when the static live load was less than the dead load stress of opposite character.

Lateral forces from trains and wind of 1 100 lb per lin ft on the loaded chord, plus 10% of the specified train load on one track, and 400 lb per lin ft on the unloaded chord, all considered as moving loads, were used.

Longitudinal forces were determined by providing for the effect of starting and stopping trains with a coefficient of friction of 20% on engine drivers and of 10% on the remainder of the train. Members subject to stress reversal were proportioned to resist the maximum stress of either character. The permissible maximum unit stresses used were as follows, in pounds per square inch:

Axial tension on net section (silicon steel)........	24 000
Axial compression on gross section of columns (silicon steel)	$\dfrac{24\,000}{1 + \dfrac{l^2}{9\,000r^2}}$
with maximum of..........................	20 400
in which, l is the length of the member, in inches, and r is the least radius of gyration	
Direct compression on steel castings.............	18 000
Bending:	
In extreme fibers of rolled shapes, built sections, and girders, net section (silicon steel).	24 000
In extreme fibers of pins (silicon steel).......	36 000
Shearing:	
Webs, gross section (silicon steel)...........	15 000
Shop-driven rivets (rivet steel)..............	12 000
Field-driven rivets, fitted bolts, and pins (rivet steel)	10 000
Bearing:	
Shop-driven rivets, projected diameter (rivet steel)	24 000
Field-driven rivets, fitted bolts, and pins, projected diameter (rivet steel).............	20 000
On masonry or concrete....................	600
Expansion rollers per linear inch (silicon steel).	600d
in which, d is the diameter of the roller, in inches.	

Proportioning of Parts.—The proportioning of the various parts and all details were made generally consistent with the Specifications for Design and Construction of Steel Railway Bridge Superstructures.[4]

Method of Design.—The theory of elastic deformation was used in determining all stresses. For convenience in determining conditions of loading required to produce maximum stresses and the stresses themselves, influence lines (Fig. 5) were used. The first set of influence lines prepared for the end and intermediate reactions were based upon the assumption that the moment of inertia of the truss throughout its length would be constant.

[4] Final Report of the Special Committee on Specifications for Bridge Design and Construction, *Transactions*, Am. Soc. C. E., Vol. LXXXVI (1923), p. 471 *et seq.*

Members of the trusses were proportioned for the stresses resulting from these reactions. By use of the theory of elastic weights, adjustments were then made in the influence lines to give effect to the variation of the moment of inertia which actually prevails, and the adjusted influence lines as shown on Fig. 5 were used in the final design. Table 1 shows a comparison of the

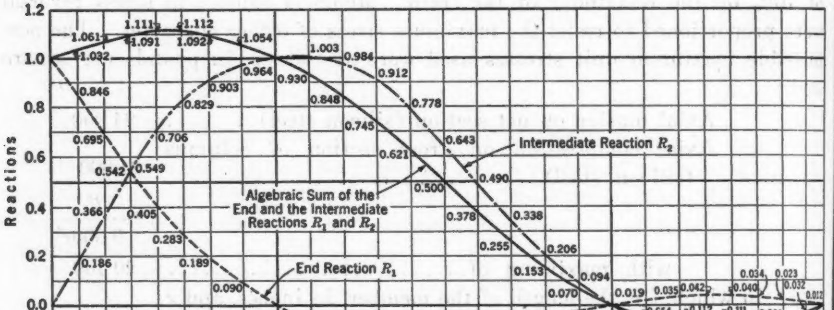


FIG. 5.—INFLUENCE LINES (FOR PANEL POINTS SEE FIG. 1)

ordinates of the influence lines based on constant moment of inertia with corresponding ordinates of the adjusted curves.

Erection stresses in no case governed the design of any member although they were based on cantilever erection of the middle span with no falsework

TABLE 1.—COMPARISONS OF INFLUENCE LINES

Panel Point No. (See Fig. 5)	END REACTION, R_1		INTERMEDIATE REACTION, R_2		Panel Point No. (See Fig. 5)	END REACTION, R_1		INTERMEDIATE REACTION, R_2	
	Constant moment of inertia	Variable moment of inertia	Constant moment of inertia	Variable moment of inertia		Constant moment of inertia	Variable moment of inertia	Constant moment of inertia	Variable moment of inertia
0......	1.000	1.000	0.00	0.00	24.....	—0.109	—0.112	0.488	0.490
2......	0.853	0.846	0.164	0.186	22.....	—0.082	—0.083	0.346	0.338
4......	0.714	0.695	0.324	0.366	20.....	—0.051	—0.053	0.211	0.206
6......	0.583	0.542	0.476	0.549	18.....	—0.024	—0.024	0.091	0.094
8......	0.452	0.405	0.616	0.706	16.....	0.0	0.0	0.000	0
10......	0.332	0.283	0.741	0.829	14.....	0.016	0.019	—0.060	—0.073
12......	0.223	0.189	0.846	0.903	12.....	0.024	0.035	—0.091	—0.127
14......	0.096	0.090	0.947	0.964	10.....	0.025	0.042	—0.098	—0.154
16......	0.0	0.0	1.000	1.000	8.....	0.024	0.040	—0.094	—0.151
18......	—0.070	—0.073	1.006	1.003	6.....	0.021	0.034	—0.079	—0.125
20......	—0.114	—0.136	0.955	0.984	4.....	0.015	0.023	—0.056	—0.084
22......	—0.136	—0.167	0.872	0.912	2.....	0.008	0.012	—0.029	—0.044
24......	—0.138	—0.167	0.760	0.788	0.....	0	0	—0.000	0
26......	—0.130	—0.143	0.629	0.643					

between the channel piers. (Stresses, caused by erection plant, 25% in excess of dead plus live load stresses were permissible.) Therefore, this method of erection did not necessitate the addition of any steel to that required in the finished structure for resisting dead, live, impact, and wind loads.

The spans of the bridge were cambered so that the center line of the bottom chord would not sag below a line parallel to the normal grade line,

as shown on Fig. 6, when the entire bridge or each span thereof is subjected to one-half the specified live load. This camber was obtained by correcting the lengths of the top and bottom chord members in accordance with the deformations produced in them and fabricating the web members to their geometrical lengths in the resulting truss figure. The trusses could thus be

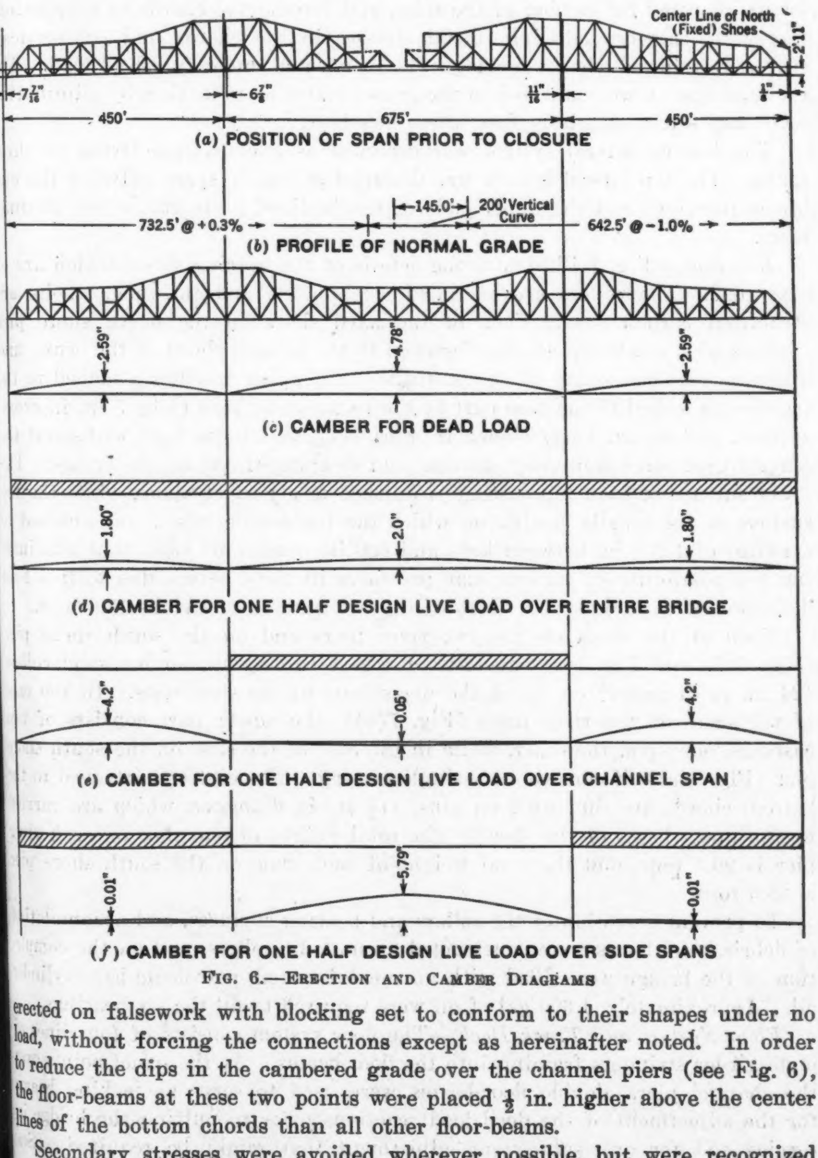


FIG. 6.—ERECTION AND CAMBER DIAGRAMS

erected on falsework with blocking set to conform to their shapes under no load, without forcing the connections except as hereinafter noted. In order to reduce the dips in the cambered grade over the channel piers (see Fig. 6), the floor-beams at these two points were placed ½ in. higher above the center lines of the bottom chords than all other floor-beams.

Secondary stresses were avoided wherever possible, but were recognized and provided for in the design where their complete elimination was not

feasible. It is evident that without special provision the heaviest secondary stresses would occur in the bottom chords and inclined posts framing into the connections at the panel points over the channel piers. Secondary stresses in the bottom chords at these points were reduced materially by making the sub-verticals on each side of these panel points $\frac{1}{2}$ in. shorter than the detailed length required for camber of the span, and forcing the chords to connections therewith, thereby inducing initial stresses in the chords and sub-verticals that were relieved when the bridge was swung and under full dead load. The inclined posts were connected to the gusset-plates by pins, thereby eliminating secondary stresses entirely from these members.

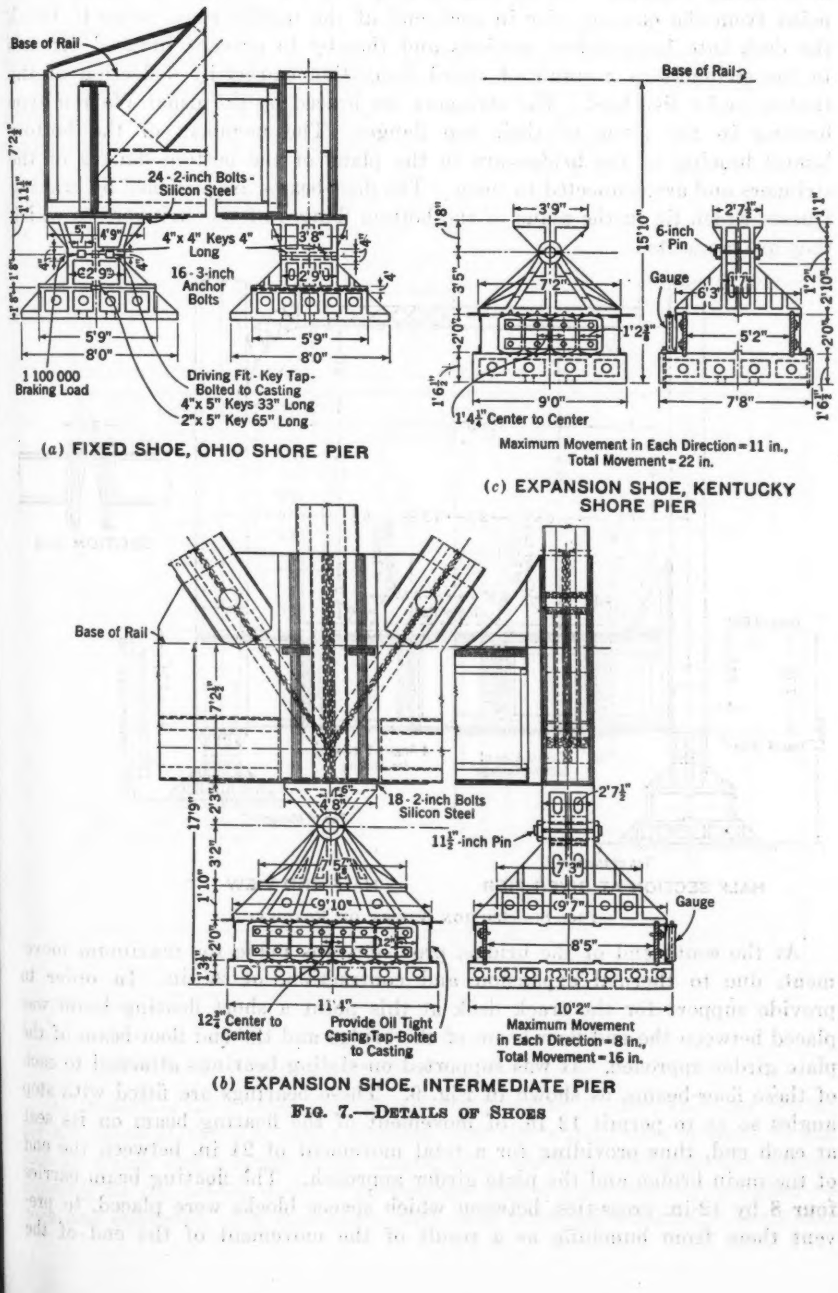
The bottom lateral system was designed as a continuous truss of three spans. The top lateral system was designed as simple spans between the end hip connections and the upper ends of the inclined posts at the two channel piers.

Bearings.—Fig. 7 illustrates the details of the bearing shoes, which are of cast steel. The rollers are of rolled steel billets and the anchor-bolts are structural carbon steel. Each of the fixed shoes on the north shore pier consists of a single top casting fastened to the bottom chord of the truss, and a bottom part consisting of two castings. The upper castings attached to the trusses are locked to the base part by the rectangular keys (4 by 5 in. in cross-section, and 33 in. long) shown in Fig. 7(a), and these keys withstand the longitudinal force caused by starting and stopping trains on the bridge. The lower surface of each top casting is finished to a perfect plane, while the top surface of the middle casting on which the top casting rests, was finished to a radius of 1 200 in. between keys and 600 in. outside of keys, thus eliminating the possibility of serious edge pressures in the castings due to live load deflection of the side span. Each of the two fixed shoes weighs 18.8 tons.

Each of the shoes on the two river piers and on the south shore pier (Fig. 7(b) and Fig. 7(c)) consists of a base casting supporting steel rollers (24 in. in diameter) on which the upper part of the shoe rests. In the case of the shoes on the river piers (Fig. 7(b)), this upper part consists of two castings, one upon the other, while in the case of the shoe on the south shore pier (Fig. 7(c)), it consists of a single casting. The castings attached to the bottom chords are supported on pins, $11\frac{1}{2}$ in. in diameter, which are carried by the upper bases of the shoes. The total weight of one shoe on each river pier is 96.5 tons, and the total weight of each shoe on the south shore pier is 38.5 tons.

To prevent corrosion of the rollers and bearing surfaces, and accumulation of débris, steel boxes were constructed around the rollers, and on the completion of the bridge were filled with an unadulterated, petroleum-base cylinder oil. Approximately, 1 600 gal of oil were required to fill the six bearings.

Floor System and Track Deck.—The floor system consists of four lines of plate girder stringers framing into the floor-beams. At the panel points over the channel piers, double floor-beams were used to serve as jacking beams for the adjustment of the dead load reactions prior to putting the bridge in service and for any subsequent adjustment that might be required. (See Fig. 8.) The bottom flanges of each of these floor-beams and those at the



(*a*) FIXED SHOE, OHIO SHORE PIER

(*c*) EXPANSION SHOE, KENTUCKY SHORE PIER

(*b*) EXPANSION SHOE. INTERMEDIATE PIER

FIG. 7.—DETAILS OF SHOES

ends of the bridge were reinforced to provide seats for the application of the jacking load under the center lines of the exterior stringers. These double floor-beams, together with sliding bearings for stringers at the sixth panel point from the channel pier in each end of the middle span, serve to break the deck into independent sections and thereby to prevent induced stresses in the stringers as a result of chord elongation caused by deflection of the trusses under live load. The stringers are braced by the usual Warren type bracing in the plane of their top flanges. The members of the bottom lateral bracing of the bridge are in the plane of the bottom flanges of the stringers and are connected to them. The floor-beams are stiffened by braking trusses which lie in the plane of the bottom flanges of the stringers to which they are connected.

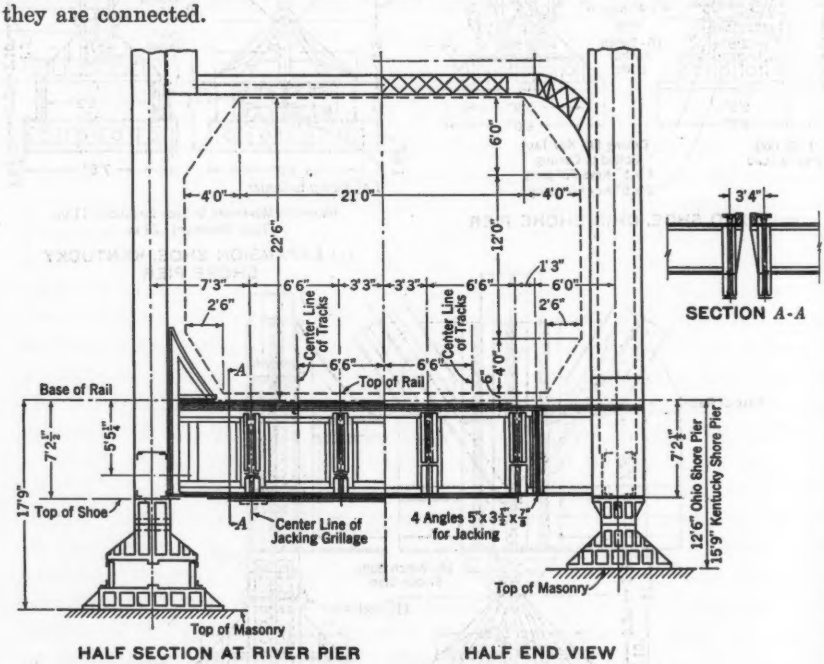


FIG. 8.—SECTION VIEWS OF ROADWAY

At the south end of the bridge, provision was made for maximum movement, due to thermal expansion and contraction, of 22 in. In order to provide support for the track deck at this point a short floating beam was placed between the end floor-beam of the bridge and the end floor-beam of the plate girder approach. It was supported on sliding bearings attached to each of these floor-beams, as shown in Fig. 9. These bearings are fitted with stop angles so as to permit 12 in. of movement of the floating beam on its seat at each end, thus providing for a total movement of 24 in. between the end of the main bridge and the plate girder approach. The floating beam carries four 8 by 12-in. cross-ties, between which spacer blocks were placed, to prevent them from bunching as a result of the movement of the end of the

bridge. The track deck consists of 8 by 10-in. creosoted cross-ties, 130-lb T-rail, guard-rails, and a plank walkway between the tracks. The rails were laid continuous over the bridge and approaches, with no special frog or other device at the south end of the bridge to provide for expansion movement. The superstructure contains 16 275 000 lb of silicon steel, exclusive of cast-steel shoes and rollers, the aggregate weight of which is 1 000 000 lb.

Erection of the Superstructure.—The side spans were erected on falsework constructed on piles driven in the river. The middle span was erected by cantilevering over both channel piers to closure at its center.

All the floor-system material for the parts of the two side spans over the north and south river banks was shipped to the site by rail and erected on the falsework by means of locomotive cranes operating on the bridge deck. Arrangements had been made for the Railroad Company to complete the viaduct approaches to the ends of the bridge before the erection contractor began on the floor system, and this greatly facilitated the work.

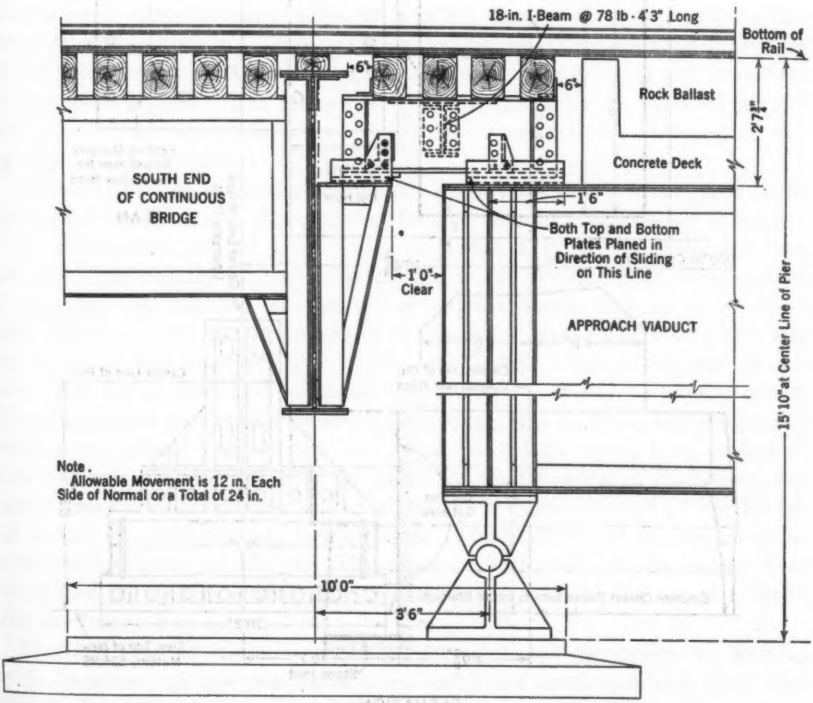


FIG. 9.—SECTION OF EXPANSION JOINT AT SOUTH END OF BRIDGE

All materials for trusses, bracing, center-span floor system, and floor system for the parts of the side spans situated over the water, were shipped to the site by water and erected from barges with the help of locomotive cranes running on the bridge deck. The heaviest single piece handled weighed 34 tons and consisted of a floor-beam with four stringers attached. Both side

spans were erected simultaneously. There was a time, in the early fall of 1928, when serious anxiety was felt for the safety of the south side span, due to delay in the completion of the south channel pier. It appeared that in order to prevent subjecting the span on falsework to the hazards of the

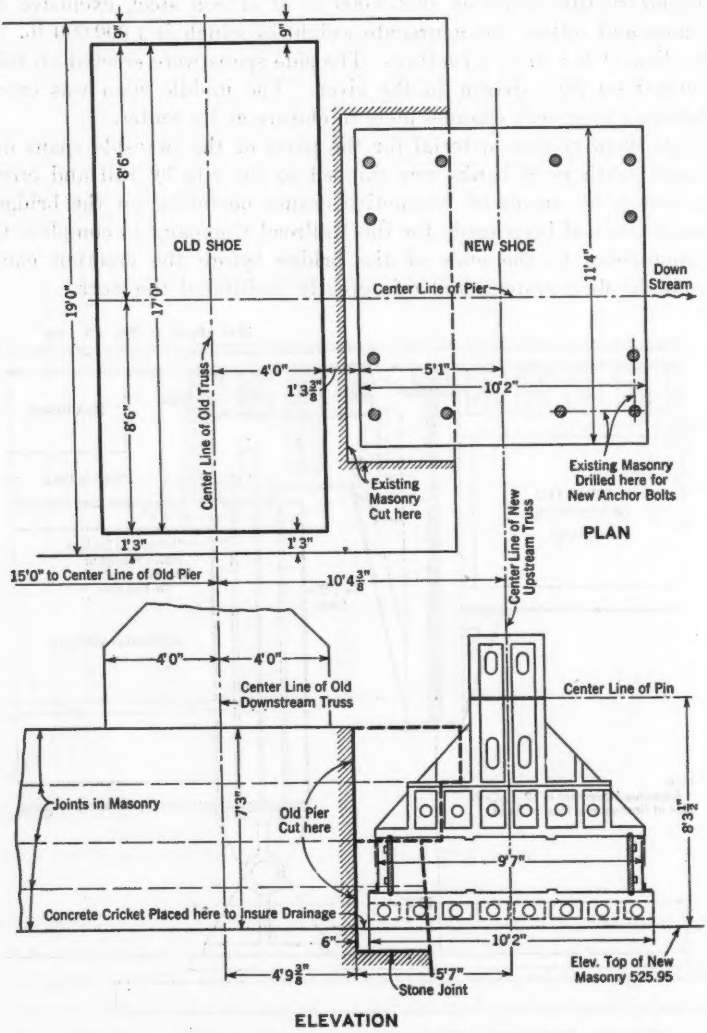


Fig. 10.—Details of Cut Made in Old Masonry Top Stem of South River Pier to Provide Room for New Shoe

high-water months, including swift flood currents, and floating ice and débris, that either the span must be dismantled and stored through the winter or the steel bent or tower, previously described under "South River Pier," on which to land the span, must be erected. The latter plan was adopted, and

the tower was fabricated but, as previously stated, its erection was not required. As a precaution, heavy timber piles were driven up stream of each bent of the falsework, and the falsework was lashed to them by steel wire cables in such a manner as to moor it against the force of a flood and also to protect it to some extent from floating débris. The span was completed and swung without mishap.

An interesting detail in connection with the erection of this side span was that caused by the necessity of removing a part of the top stem of the old south river pier enough to clear the up-stream shoe of the new bridge (see Fig. 10). The importance and volume of railroad traffic carried on the old bridge during the erection of the new bridge made it impossible to abandon the use of either track even for a short time. As the part to be cut from the down-stream end of the top stem of the old pier extended to a vertical plane tangent to the down-stream end of the shoe of the old bridge, and as the structural value of the masonry of the old pier was more or less unknown (even though grout had been pumped into it), some anxiety was felt for the safety of the southbound track. The operation was accomplished safely by careful work in removing only as much of the old pier stem as was necessary to clear the new shoe. The masonry of the inshore and river faces of the old stem was left in place and served as buttresses against the shearing of the top edge of the old pier.

While the bridge was being erected the fixed shoes were omitted, and jacks were placed on the north shore pier under the ends of the trusses. This end of the bridge was shifted $\frac{1}{8}$ in. south and held 2 ft 11 in. below its final position. The centers of the shoes on the north channel pier were shifted $\frac{11}{16}$ in., and the centers of the shoes on the south channel pier were shifted $6\frac{7}{8}$ in., north of the centers of bearings on their respective piers, and the rollers of the shoes on both piers were locked. The south end of the bridge was set so that the end panel point was $7\frac{7}{16}$ in. north of the center of bearing on the pier; the shoe above the rockers was omitted, and temporary blocking was placed between the chord and the rollers which were locked so as to maintain the chord 2 ft 11 in. below its final position. At closure, the bottom chord connection was made, after which the north end of the bridge was jacked up until the top chord closed. The position of the bridge prior to closure is shown on Fig. 6. The shoes at both ends of the bridge were erected and the lock-bars were removed from the rollers of the movable shoes, after the bridge had closed.

The final step in erection consisted of "weighing" the bridge by jacking up the bearings on the two shore piers until the predetermined dead load reaction (of 617 200 lb) on each pier was attained, as shown by pressure gauges on the jacks. Where necessary, shims were placed between the underside of the bottom chords and the shoe castings in order to maintain the bridge in the form producing the aforementioned dead load reaction. The position of the movable shoes was checked and found to be correct, and the bridge was ready for service.

Conclusion

The continuous bridge is a type that should receive more consideration than has been accorded it in America. It is particularly well suited to railroad service because of its great rigidity.

Acknowledgments

The United Gas Improvement Contracting Company, of Philadelphia, Pa., built the piers, and the American Bridge Company furnished and erected the steel superstructure. The bridge was designed by The J. E. Greiner Company, which served in a consulting capacity during the construction, and was built under the supervision of C. W. Johns, M. Am. Soc. C. E., and Crosby Miller, Assoc. M. Am. Soc. C. E., Chief Engineer and Bridge Engineer, respectively, of the Chesapeake and Ohio Railway Company, with Mr. G. G. Lancaster, of the Railway Company, as Resident Engineer.

Construction of the substructure was begun in July, 1927, and completed in October, 1928. Work was often interrupted by abnormally frequent and prolonged periods of high water during the spring and summer of 1928. Steel erection was begun in June, 1928, and the bridge was put in service in March, 1929.

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852
DISCUSSIONS

FORESTS AND STREAM FLOW

Discussion

By MESSRS. HERMAN STABLER, AND H. S. GILMAN

HERMAN STABLER,[60] M. AM. SOC. C. E. (by letter).[60a]—The American people are fond of hokum, and none is so prone to accept its dictates as scientists—seekers after truth—who delve in theories in the absence of substantial factual foundation. Perfect logic starting from false premises will lead to false conclusions. A consensus of scientific opinion in the United States thirty-five years ago on the subject of forests and stream flow is contained in the following excerpts from a report of a Committee (Messrs. Charles S. Sargent, Henry L. Abbott, A. Agassiz, William H. Brewer, Arnold Hague, Gifford Pinchot, and Wolcott Gibbs) of the National Academy of Sciences[61]:

"The influence of forests upon climate, soil, and the flow of water in streams has attracted much attention during the past century; but while the general consensus of opinion among experts is that this influence is potent and beneficial, the absence of exact data extending over long periods of time, and the complex nature of the phenomena involved, render it necessary to base this conclusion rather upon general considerations than upon statistics."

With this introduction the Committee painted a picture of the natural regimen of streams "replaced by destructive floods in the spring and by dry beds in the months when the irrigating flow is most needed" and of prosperous communities "depopulated" by reason of assumed destruction of forests on mountainous water-sheds of the arid regions of North America; declared that "unrestricted pasturing of sheep" must inevitably render "worthless for irrigation" streams rising in the Sierras and Southern Cascades; and concluded with the general opinion,

NOTE.—The paper by W. G. Hoyt, M. Am. Soc. C. E., and H. C. Troxell, Assoc. M. Am. Soc. C. E., was presented at the Annual Convention, Yellowstone National Park, Wyoming, July 6, 1932, and was published in August, 1932, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: September, 1932, by C. G. Bates, Esq.; November, 1932, by Messrs. J. E. Willoughby, and A. L. Sonderegger; December, 1932, by Messrs. J. C. Stevens, Harry F. Blaney, Daniel W. Mead, Ralph R. Randell, H. K. Barrows, Donald M. Baker, Ralph A. Smead, and George H. Cecil; February, 1933, by Messrs. C. W. Sopp, and W. B. Rowe; and March, 1933, by Messrs. W. C. Lowdermilk, Rhodes E. Rule, and Robert E. Kennedy.

[60] Chf., Conservation Branch, U. S. Geological Survey, Washington, D. C.

[60a] Received by the Secretary March 1, 1933.

[61] Report of the Committee Appointed by the National Academy of Sciences upon the Inauguration of a Forest Policy for the Forested Lands of the United States, to the Secretary of the Interior, May 1, 1897, Washington, Govt. Printing Office, 1897.

"That, while forests probably do not increase the precipitation of moisture in any broad and general way, they are necessary to prevent destructive spring floods, and corresponding periods of low water in summer and autumn when the agriculture of a large part of Western North America is dependent upon irrigation."

It is little wonder that, with such backing of scientific opinion—even though based admittedly "rather upon general considerations than upon statistics," aided and abetted by the natural fondness of most people for trees and the undeniable value of forests as sources of wood and lumber—nation-wide propaganda for forests as conservators of water and regulators of stream flow has taken firm hold on the imagination of the people and that it has become so firmly rooted in the popular mind as to be regarded as more or less axiomatic. Messrs. Hoyt and Troxell are to be congratulated for bringing before engineers statistical results in succinct form on the basis of which the volumes of propaganda, written and preached to the American public, may be adjudged. These statistical results, obtained in the very region chiefly considered by the Committee of the National Academy of Sciences, show that the Committee erred in assuming that loss of vegetative cover would produce dry beds of streams in summer and render them worthless for irrigation, and that claims heretofore made in behalf of forests on the basis of "general considerations" are clearly fallacious in some areas and, in the absence of statistical data to the contrary, must be assumed to be equally fallacious elsewhere.

The writer would be particularly interested to see the results of an experiment, such as that conducted at Wagonwheel Gap, undertaken in a humid region and under conditions most favorable to maintenance or betterment of low-water flow by vegetation. Such an experiment would substantially determine whether the principles set forth by Messrs. Hoyt and Troxell are universally applicable. Until such an experiment, conducted under trustworthy auspices, proves the contrary, one must assume universal applicability of these principles because of the wide diversity of conditions covered by their studies.

No one can be more interested than the hydraulic engineer in the actual facts of the case, and it is hoped that engineers will assist in accumulating a mass of statistical data relative to effects of vegetation on stream flow, that will furnish conclusive evidence applicable to a great variety of conditions. Engineers, however, are not by nature missionaries or propagandists. It will remain for some other group to educate the public. Is it too much to hope that those who have been most active in spreading doctrines now found to be fallacious will be equally zealous in disseminating the results of trustworthy statistical studies?

As a supplement to the studies of Messrs. Hoyt and Troxell mention should be made of another attempt to determine the effect of vegetation on run-off and erosion. This study was conducted on grass- and weed-covered slopes of the Wasatch Mountains, in Utah, and is reported by Mr. C. L. Forsling.[63]

[63] "A Study of the Influence of Herbaceous Plant Cover on Surface Run-Off and Soil Erosion in Relation to Grazing on the Wasatch Plateau, Utah," by C. L. Forsling, *Technical Bulletin No. 220*, U. S. Dept. of Agriculture, Washington, D. C., March, 1931.

The published results of the experiment show wide discordances in precipitation, run-off, and erosion for different observations. Amount and distribution of rain, intensity of precipitation, condition of surface as to moisture, temperature, marked differences in topographic and channel characteristics of the two areas, etc., all serve to obscure whatever influence vegetative changes may have exercised.

After extensive independent study of all results of the experiment the writer has concluded that summer storms, which produce most of the destructive run-off and erosion in the Wasatch region, offer the most instructive field for comparison with the results found by Messrs. Hoyt and Troxell. He, therefore, presents herewith a brief description of the conditions of the experiment and of the results of his study of the data relating to important summer rainfall.

Two adjacent areas of about 10 acres without perennial run-off were selected for study, Area *A* having a vegetative cover of 16% and Area *B*, a cover of 40 per cent. Collecting basins were constructed to catch all run-off and sediment, and rain-gauges were set up to measure precipitation amounting to about 30 in. per year. Observations were continuous from 1915 to 1929. Area *B* was maintained with 40% cover throughout the period by light grazing of cattle or sheep once or twice each summer. Area *A* had 16% cover through 1919, this being maintained by grazing as on Area *B*; increase to 40% was attained from 1920 to 1923 or 1924, by elimination of all grazing and the sowing of grass seed (wheatgrass and bromegrass) near the upper end of the area; and this higher value was maintained until the end

TABLE 21.—DATA RELATING TO JULY AND AUGUST STORMS.

Date of storm	Average precipitation, in inches	Run-Off, in Inches		Sediment, in Cubic Feet	
		Area *A*	Area *B*	Area *A*	Area *B*
August 2, 1916	0.34	0.0105	0.0064	15.9	5.1
August 5, 1916	0.26	0.0122	0.0121	8.1	4.8
August 9, 1917	1.04	0.0997	0.0156	67.2	6.4
July 9, 1918	0.27	0.0215	0.0002	26.6	0.2
July 10, 1918	0.21	0.0121	0.0002	6.1	0.2
July 13, 1918	0.72	0.0612	0.0032	60.1	2.0
July 12, 1919	0.58	0.0081	0.0004	6.9	Trace
August 8, 1920	0.84	0.1532	0.0248	79.0	28.5
August 25, 1920	0.44	0.0206	0.0074	17.1	7.7
July 15, 1921	0.62	0.0275	0.0009	11.1	0.6
July 17, 1921	1.25	0.2325	0.0534	82.7	23.1
August 15, 1921	0.46	0.0296	0.0016	12.3	0.6
August 23, 24, 1921	0.79	0.0405	0.0201	16.0	12.5
August 25, 1921	0.08	0.0043	0.0003	1.1	0.3
August 26, 1921	0.20	0.0171	0.0171	8.6	11.5
August 27, 1921	0.08	0.0001	0.0024	Trace	0.3
August 28, 30, 1921	0.56	0.0308	0.0273	19.4	11.5
August 1, 1922	0.70	0.0432	0.0059	23.7	2.9
August 31, 1922	0.36	0.0293	0.0128	23.6	18.5
August 13, 1923	0.86	0.1627	0.0572	75.1	25.1
July 27, 1924	0.84	0.0007	0.0007	Trace	Trace
July 4, 1925	0.20	0.0064	0.0053	1.1	Trace
July 3, 1926	0.48	0.0017	0.0011	Trace	Trace
July 4, 1926	0.36	0.0140	0.0093	11.7	2.4
July 21, 1927	0.70	0.3270	0.1326	55.5	30.0
July 27, 1927	0.56	0.0895	0.0735	28.9	13.2
July 28, 1927	0.22	0.0045	0.0029	0.8	0.3
August 5, 1929	0.63	0.0299	0.0017	14.7	0.5

of 1929, being kept down by grazing very lightly by cattle in September, 1925, and moderately by sheep once in late September or early October of 1927, 1928, and 1929.

Table 21 gives the statistical data as recorded by Forsling for July and August storms that produced run-off on both areas and that showed variation of not more than 25% in precipitation on the two areas.

By simple arithmetical operations, the progressive accumulations of precipitation, run-off, and sediment, the progressive accumulations of excess run-off and sediment from Area *A* over run-off and sediment on Area *B*, and the ratios (Area *A* to Area *B*) of progressive accumulated run-off, sediment, and sediment per inch of run-off, were determined from the data in Table 21 and are presented in Figs. 14, 15, and 16.

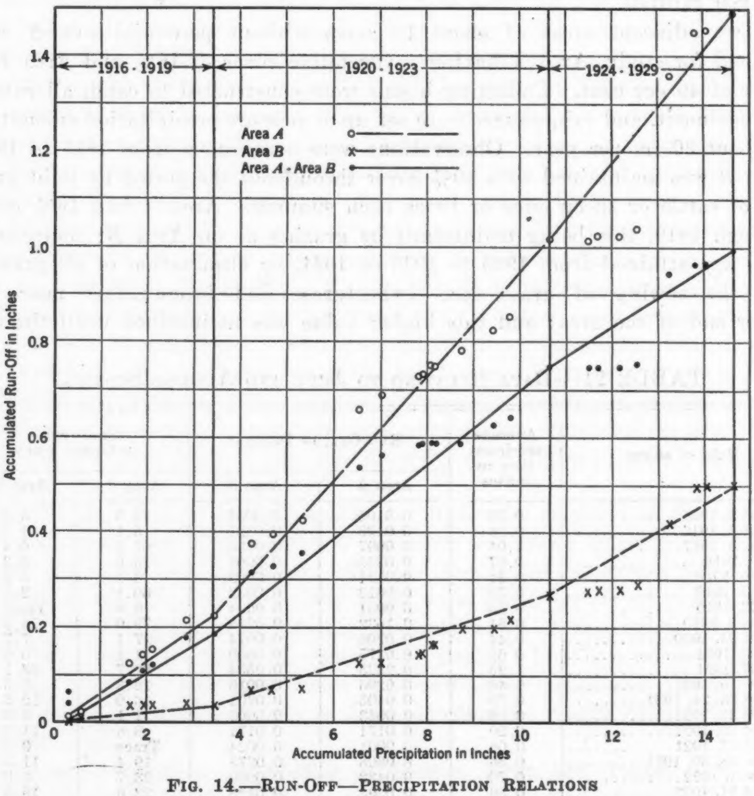


FIG. 14.—RUN-OFF—PRECIPITATION RELATIONS

Fig. 14 indicates that, relative to precipitation occurring in summer storms, Area *B* (vegetation constant) produced run-off at a higher rate in the second period (1920-1923) than in the first period (1916-1919), and at a still higher rate in the third period (1924-1929). Area *A* (vegetation variable) shows a similar trend although the rate of run-off in the third period is very slightly greater than in the second. The excess of run-off from Area *A*

over run-off from Area B shows very little change in rate. The inference to be drawn from these curves is that vegetational changes on Area A affected run-off but little—less, indeed, than other factors.

Fig. 15 indicates that, relative to precipitation occurring in summer storms, sediment was washed from Area B (vegetation constant) at a higher rate in the second and third periods than in the first, the rate being slightly less in the third period than in the second. Sediment was washed from Area A (vegetation variable) at practically the same rate in the first two periods, but at a slower rate in the third period. The excess of sediment washed

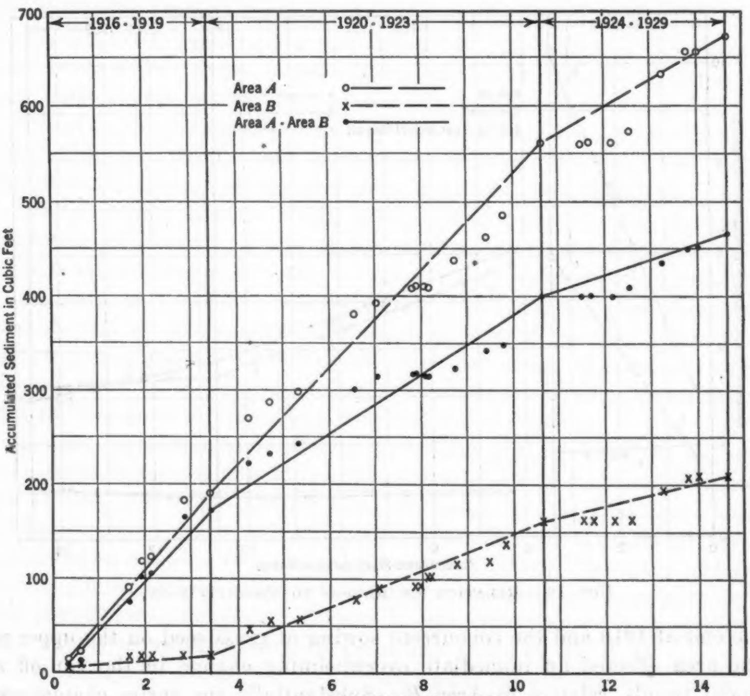


FIG. 15.—SEDIMENT—PRECIPITATION RELATIONS

from Area A over sediment washed from Area B was at a lower rate in the second period than in the first and at a lower rate in the third period than in the second. These trends strongly suggest that as vegetative cover on Area A was increased sediment removal was decreased. Doubt on this suggestion is cast by the fact that Area B, with constant vegetation, showed a greater rate of change than did Area A and that most of the decrease in excess of sediment (Area A over Area B) is due to the marked increase in sediment removal from Area B during the second and third periods.

Fig. 16, which deals only with relative results and, therefore, depends wholly on the reliability of Area B as a constant or "blank" for purposes of comparison, indicates, relative to precipitation occurring in summer storms,

a striking reversal of trend both for ratios of run-off and for ratios of sediment removal. The ratio of sediment removal per inch of run-off, nearly constant at 1.7 during the first period, drops abruptly to another approximate constant, only about one-half as large, at the beginning of the second period, and changes but little from that point on to the end of the experiment. The third period shows, with respect to the second, a slight increase in the run-off ratio ($A : B$), a slight but somewhat greater increase in the sediment ratio ($A : B$), and a slight increase in the sediment ratio ($A : B$) per inch of run-off. These trends strongly suggest that elimination of grazing on Area A

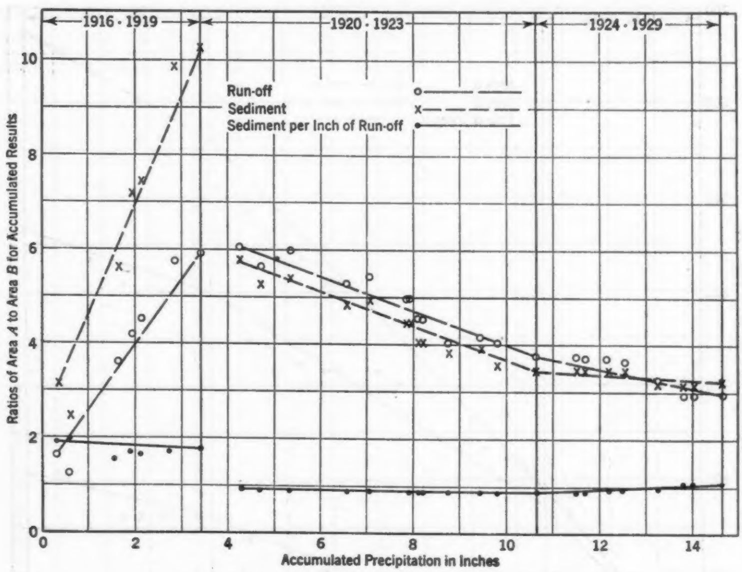


FIG. 16.—RELATION OF RATIOS TO PRECIPITATION

at the end of 1919 and the concurrent sowing of grass seed on the upper part of the area effected an immediate overwhelming change in the run-off and sediment trends, relative to Area B. Substantially the entire change seems to have occurred between July, 1919, and August, 1920. Yet Forsling's report states that "although grazing was discontinued in 1920, there was no change in the vegetation in that year." If this statement, surprising in view of the sowing of grass seed, can be accepted at face value one must look to the presence or absence of grazing animals rather than to any change in vegetation for the reversal of trends. This thought gains some strength from the observation that, in the third period, after grazing on Area A had been resumed in part, there is an indicated tendency toward return to conditions of the first period. However, bearing in mind that most of the trends are due to variations on the supposed constant Area B (which, as noted, is not a fixed quantity), all conclusions reached must be considered as tentative if not speculative.

If the curves had been constructed with time elements instead of precipitation as abscissas, the slopes of all lines in the second period would be steeper and those in the third period flatter with reference to the slopes of lines in the first period. This follows directly from the record of 3.42 in. of rainfall in the four summers of the first period, 7.24 in. of precipitation in the four summers of the second period, and 3.99 in. of precipitation in the six summers of the third period.

The writer is inclined to draw the following tentative conclusions from his studies of the Wasatch Plateau experiment in the light of the study by Messrs. Hoyt and Troxell:

1.—On the steep grass and weed-covered mountain sides of the Wasatch Plateau, Utah, wide changes in vegetative cover and in grazing practices (*a*) produce little change in the rate of run-off from summer storms, slightly more run-off with more grazing and with less vegetation being indicated; and (*b*) affect notably the rate of sediment removal from summer storms, more erosion with more grazing and less vegetation being indicated.

2.—Under the conditions of soil, relief, and climate affecting the experimental areas, measures that will restore and maintain the maximum plant cover for grazing purposes will also insure adequate water-shed protection.

3.—A cover of grass and weeds, grazed under careful regulation, may afford protection to soil as satisfactorily and more profitably and deplete water supply less than a cover of useless brush. Relative effects of grass and weeds, brush, and trees on water supply and soil protection should be studied to afford a reliable guide in administration and use of lands.

4.—If wide variations in vegetative cover and grazing practices on steep mountain sides where the annual precipitation is about 30 in. produce changes in run-off and erosion no greater than indicated by the Wasatch Plateau experiment, neither variations artificially producible in the annual forage crop of about an ounce to the square yard on the relatively flat-lying and arid unreserved public domain (170 000 000 acres), nor the presence or absence of a stock population sufficient to harvest it, can affect appreciably problems of water supply or sedimentation of the arable lands of the West.

5.—The study, in general, tends to corroborate the conclusions reached by Messrs. Hoyt and Troxell and to extend their application.

H. S. Gilman,[63] Esq. (by letter).[63a]—It is quite possible that there are areas of water production and use, throughout the world, that will be greatly benefited by the findings and recommendations embodied in this paper. It is also possible that the authors' recommendations as to water-shed management will be adopted in field practice in an attempt to increase water yield. The result of actual field practice, if accompanied by close observations of all factors in water yield, may finally settle this old controversy of the beneficial or detrimental influence of forests upon stream flow and water yield. It is a surprising coincidence, however, that the observations upon which this discovery and findings were based, occurred on areas in Southern California, where, because

[63] Civ. Engr.; Member, California State Board of Forestry, San Dimas, Calif.

[63a] Received by the Secretary February 27, 1933.

of a combination of natural conditions, water-shed protection is popularly supported and practiced more intensively possibly than in any other area. Steep mountain slopes, a shattered rock formation, a layer of soil, a covering of scrub forest native to a semi-arid region, the year's rainfall occurring for the most part within a period of ninety days, few and costly reservoir facilities, a few miles of travel for the water from the mountains to the ocean—this briefly sets forth the water-producing machinery, and the authors recommend eliminating the vegetative cover and speeding up the run-off.

As manager of a mutual water company in Southern California for the past seventeen years, the writer has been in charge of diversion, production, and distribution of water for irrigation and domestic use, in an area of water production and use very similar to the areas of Santa Anita and Fish Canyons compared in this paper.

As is the case in most parts of Southern California, the annual water crop is produced from two sources—diversion of surface flows of the mountain canyons, and pumping from the alluvial basins beneath the plains and valleys. The surface gravity flows are unreliable in quantity as a source of supply, due to the seasonal variations in rainfall. The basins are more reliable as a constant source of supply, and act as an equalizing reservoir to carry over the years of deficient rainfall. For the past seventeen years, 20% of the Company's supply has come from gravity water and 80% has been pumped. Therefore, basin water, although higher in cost of production, is the most valuable because of its greater dependability.

Many water companies and agencies, recognizing the value of underground storage, have expended large sums of money in artificial works to increase percolation of storm waters over the débris cones at the mouths of the canyons for the purpose of recharging the basins.

In 1919, a forest fire denuded 50% of the slopes of the water-shed tributary to the area from which the Company produces its water, and the writer has direct knowledge of the effect of cover disturbance upon actual water production as opposed to a theoretical study which touches on one phase only, that of surface flow.

Water conditions in Southern California are such that maximum recovery of the annual precipitation must be made in both diversion of stream flows and through the pumping of underground waters. Any practical solution or suggestion as to water-shed or basin management that will increase the yield will be quickly applied to the problem by those in charge of water production. Arguments as to the benefits of denudation are as old as the hills themselves. The paper contributes nothing new as far as authentic data are concerned; and it overlooks, or sets aside as being of no consequence, certain factors that control water production, in all its phases, in this area. It is simply another instance of a plausible theory that will not stand the test of field practice.

The application of Mt. Wilson rainfall data as a measure of precipitation upon both the Santa Anita and Fish Canyon areas to develop a formula for comparison of actual as against theoretical flows, in this scientific discussion,

can scarcely be accepted as being sufficiently authentic to place in effect so radical a change in water-shed management as is recommended. Mountain rainfall records in Southern California are few and far between. Such as are available indicate great variations in seasonal storms and hourly intensity in the various areas. Variations up to 100% have been noted between stations ½ mile apart for various storms, and the picture of uniform precipitation in quantity and intensity over the areas reported, can be based only upon a wild assumption.

The U. S. Geological Survey records of stream flow in Southern California are full of examples of great variations per square mile of water-shed run-off between adjacent or distant drainage areas, where no cover disturbances have occurred. An intensive observation of actual rainfall "catch" upon these areas under discussion might have indicated a greater or lesser increase in yield, but under the circumstances the findings are valueless.

Besides questioning the authenticity of rainfall data used by the authors, other fundamental objections may be taken in their attempt to show a beneficial increased yield in production by reasons of denudation.

In the first place, the measurement of water over a weir is not the final measurement of water yield. In practical diversion and application to use, the condition and usability of the water, as well as the relation of mountain precipitation and run-off to basin recharge, are as important as the quantity. In the paper, while reference was made to the great percentages of solids in the water, apparently no correction of weir measurements was made, and everything flowing over was counted as water to show the beneficial increase. In practice of diversion and use, however, it is necessary for those in charge to make such corrections, not on paper, but physical corrections in the field, before the water can be used for spreading purposes or delivered and sold for use. Suspended matter in water prevents percolation into the basins and is objectionable in both irrigation and domestic use. The writer found that before the 1919 fire the stream cleared within two or three days after a storm so that it was usable for any purpose. For four years after the fire, suspended matter in the water and the stream-bed flow of sand and gravel, upset the old program of diversion and use to such an extent that any benefit in increased yield as indicated by Messrs. Hoyt and Troxell (including as it does the solids), was more than offset by necessary loss of water through the sluicing of settling basins before diversion could be made. The small increase in summer flow would be a high price to pay for the loss of percolation of winter waters in recharging the more important underground supplies.

That the data used by the authors permitted findings parallel to those for Wagonwheel Gap can only be laid to accident. If reliable data had been available and if the findings resulted as reported, the recommendations would still not be practicable in Southern California in the absence of recommendations as to erosion control, desilting, and a safe and economical method of denudation.

Considering their findings that, where flood control is a major problem of a drainage area, the maintenance of an undisturbed cover is desirable, it

is difficult to visualize an area where population and irrigation demands are such that an increased yield of water is so imperative as to demand the drastic treatment of a water-shed by deliberate denudation, at the same time exposing itself, its people, and values to an increased flood threat.

Several counties in Southern California are spending many millions of dollars in the construction of engineering projects in an attempt to control or retard the normal and flood flows during the winter seasons. These projects, for the most part, are storage reservoirs in the mountains, usually constructed at high cost per acre-foot of storage. These reservoirs are expected to operate under conditions of normal water-shed cover conditions. It is questionable whether they would function under a denudation program of water-shed management, in the light of the authors' showing of a great percentage of increased run-off during the period of normal heavy flow. Furthermore, the economic life of a high-cost reservoir should be a matter for consideration in the light of the great increase of eroded material in the flow. The great percentage of increased yield during a period of heavy normal yield under these conditions, raises the question as to whether or not the increase is beneficial, while the lesser increased yield during the period of normal flow, or no flow, amounts to a surprisingly small quantity of actual water when computed in dollars and cents.

It is the writer's opinion that men in charge of actual water production and diversion of water in Southern California are as strongly in favor of water-shed protection as ever.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

TESTS FOR HYDRAULIC-FILL DAMS

Discussion

By Messrs. D. P. Krynine, and M. M. O'Shaughnessy

D. P. Krynine,[27] M. Am. Soc. C. E. (by letter).[27a]—The publication of the test procedure during the construction of the hydraulic-fill dam at Cobble Mountain has rendered valuable services not only to engineers interested in that particular kind of work, but to all who are engaged in soil mechanics research.

The writer was much interested in the comparison of elutriator and hydrometer methods described in the paper. The curves in Fig. 6 are quite close to each other; moreover, there is no possibility of giving definite preference to either of them. Since this is so and since the hydrometer is not expensive, requiring only a short time for the performance of the test, a question arises as to whether the elutriator should be recommended for future tests of the same character.

Relative to the slight difference between the curves in Fig. 6, it should be noted that the gradation curves for the hydrometer in the lower limits, show a higher percentage of finer particles than the elutriator. According to the author's statement, logically, it should be the reverse, as some of the fine particles may go down with the larger ones.

In the writer's opinion, since the deviation of the curves in Fig. 6 is systematic, there should be a rational cause therefor. Probably the explanation is to be found in the conventionality of the position of the center of buoyancy. Although the curves in Fig. 4 are determined experimentally, they refer to a uniform solution or suspension, while what is dealt with in an actual case, is a non-uniform soil suspension. The center of buoyancy may be determined as the center of gravity of the displaced liquid. During the experiment, coarser particles tend to occupy lower positions than the fine ones,

NOTE.—The paper by Harry H. Hatch, M. Am. Soc. C. E., was presented at the Joint Meeting of the Irrigation and Power Divisions, Yellowstone National Park, Wyoming, July 7, 1932, and published in October, 1932, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: January, 1933, by Charles H. Paul, M. Am. Soc. C. E.; February, 1933, by Joel D. Justin, M. Am. Soc. C. E.; and March, 1933, by Messrs. Jeptha A. Wade, and Stanley M. Dore.

[27] Research Assoc. in Soil Mechanics, Yale Univ., New Haven, Conn.

[27a] Received by the Secretary March 8, 1933.

and, in a general case, the density of the suspension increases toward the bottom. Hence, the center of buoyancy of a non-uniform soil suspension should be lower than that of a uniform one. Owing to this fact, the actual abscissas in Fig. 4 are probably under-estimated. Therefore, this should also be true of the diameter of particles that is to be computed, using Equation (8). In other words, in a general case, a certain over-estimation of the amount of the fines seems to be inevitable.

From a practical point of view the variation shown in Fig. 6 is quite negligible (perhaps 1 or 2% by weight). An ideal "size distribution curve" cannot be obtained, and it is not worth while to spend time in refined determinations. In this connection, the writer does not see very clearly why the curves of Fig. 4 should be traced so accurately. Those curves take into account one factor only, namely, the peculiarities of a given hydrometer; but they do not pay attention to the other factor, which is the non-uniformity of the soil suspension in the cylinder. In addition, the degree of non-uniformity is changing during the entire experiment. Perhaps simple straight lines could be substituted for those curves with the same degree of accuracy as far as the final results are concerned.

The author assumes that the distance of fall of a particle in the experimental cylinder is equal to the distance from the surface of the liquid to the center of buoyancy. The writer does not possess concrete data with which to contradict this statement, but it should be noted that some investigators assume a smaller value. The U. S. Bureau of Public Roads, for instance, takes the assumed distance of fall as 0.42 times the distance from the surface of the suspension to the elevation of the bottom of the hydrometer.[28]

Different fractions of a soil may possess different specific gravities; this is especially true of fine particles. Consequently, the center of buoyancy may be displaced. This fact has never been studied.

The author compares the hydrometer and the elutriator. It would be better, perhaps, to compare the hydrometer and the pipette method; actually, the latter may be considered as a more or less reliable standard. Another test which, apparently, should not be used hereafter, is the turbidity test, because, as the author states, it has lost its value since the introduction of the hydrometer method.

In discussing the penetration test the author states that "by applying the total load at the beginning of the test, the pipe would penetrate farther into the core than by applying the same total load in fractions at different intervals." The difference involved was a penetration of 2 or 3 ft less. This is a notable fact which is analogous, however, to the increase in resistance of a pile after rest, especially in impermeable soils. This fact is well known to both practical pile-drivers and soil-mechanics theorists. The author's Equation (2) is merely local and cannot be generalized.

Important and highly interesting experiments were made by the author in determining the coefficient of friction. The curves in Fig. 10 show clearly that the coefficient of friction of a soil is not constant, but that it depends

[28] Reports on Subgrade Soil Studies, Reprinted from *Public Roads*, Vol. 12, Nos. 4, 5, 7, and 8, p. 69.

on the outside pressure. Since the experimental data are not plotted in Fig. 10, it is difficult to say whether or not these curves tend asymptotically toward infinity when the outside pressure vanishes. This tendency is shown in Fig. 10 and is also required by Equation (18). Another solution would be an intersection of the axis of ordinates at determined finite heights, a result reached[29] by Charles Terzaghi, M. Am. Soc. C. E. The right side of Fig. 10 furnishes very consistent results which coincide with those of other investigators. For instance, Terzaghi[29] gives the value 0.52 for compacted Ottawa sand, the outside pressure being 10 kg per sq cm. Practically the same value is given by the author (load, 500 lb per sq in.).

Equation (1) gives the percentage of voids, based on the percentage of water by weight and the specific gravity of core material. In other branches of soil research, however, the percentage of water is always determined by dry weight. If w_0 designates the percentage of moisture by dry weight, the equation for determining the percentage of voids, would be:

$$V = \frac{100\ w_0\, s}{w_0\, s\ +\ 100} \quad \ldots\ldots\ldots\ldots\ldots\ldots (44)$$

Obviously, the results computed according to Equation (1) and Equation (44) should be the same.

Another difference is in plotting results of mechanical analysis. In practically all papers dealing with dams, the coarser particles are at the right side of the diagram, as in Fig. 8; while the "size-distribution curve" used in some other research branches is constructed exactly in the opposite manner. Soil mechanics is a young science, and small discrepancies similar to those described, should be reconciled immediately.

As to the seepage formulas, there are not only formulas, but books, written on that subject. To the writer's knowledge, there is a German work, that deals with experiments on the moisture movement in the interior of an earth dam and in which attempts are made to express the results analytically.[30] Another book printed in Russian[31] is an interesting mathematical treatise discussing seepage phenomena in earth dams.

M. M. O'Shaughnessy,[32] M. Am. Soc. C. E. (by letter).[32a]—The author is to be commended for the various devices developed in procuring information as to the texture of the core materials for this phenomenally high composite dam. A very rational method was adopted in constructing substantial toe-walls of loose rock, which are a valuable asset to any kind of an earth dam. The core material is of particularly thin section for this type of dam. The concrete toe-wall is a substantial structure, but the writer questions the wisdom of leaving such large weep-holes as 4 by 6 ft in size. A greater number of smaller weep-holes, with clean porous gravel, are far more effective.

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[29] "Handbuch der physikalischen und technischen Mechanik," Edited by F. Auerbach and W. Hort, Vol. IV, Pt. 2, p. 523, Leipzig, 1931.

[30] "Die Wasserbewegung in Dammkörper," by Ignaz Schmied, 1928.

[31] "Seepage Through Earth Dams on Impervious Foundations," by Prof. N. Pavlovsky. 1932.

[32] Cons. Engr., San Francisco Public Utilities Comm., San Francisco, Calif.

[32a] Received by the Secretary March 6, 1933.

The dam material was discharged from the mud line, the larger particles settling first, and then smaller and smaller particles until the very finest formed the core pool. The very fine particles held in suspension ultimately make up the core or water-tight section of the dam.

The Cobble Mountain Dam is the first of this type built in the East. The practice of hydraulic-fill construction has been developed successfully in the West for a great many years. This dam is a unique structure, pioneer of its particular type, and is entitled to all the success its projectors anticipate.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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DISCUSSIONS

A HISTORY OF THE DEVELOPMENT OF WOODEN BRIDGES

Discussion

By Messrs. Oren Reed, R. P. Davis and L. L. Jemison, Peter T. Landsem, J. K. Finch, A. A. Eremin, Aksel Andersen, and William G. Atwood

Oren Reed,[63] Assoc. M. Am. Soc. C. E. (by letter).[63a]—The authors are to be commended for their excellent history of the development of different types of wooden bridges, a class of engineering structure that is fast disappearing in Central and Eastern United States. Since the structures described were located in the Eastern States, the writer wishes to refer to a few wooden bridges of Indiana to show the wide range in the use of such bridges. Most of these old structures, which are still in use, are on little back roads, off the main highways, and in out-of-the-way places; but a few are on major routes, such as the multiple-span bridge on the present U. S. Route No. 50 over the East Fork of White River, near Medora, and the two-span bridge on State Road No. 52, at the edge of Rushville, over Flat Rock River.

Nearly all the early covered bridges in Indiana were of the Burr truss type. The Howe truss soon superseded the Burr truss, to a large extent, in the East, but was not commonly used in Indiana, and probably not at all before 1865. The National Road, the Michigan Road, and the Vincennes-New Albany Pike were the early important roads and most probably the ones on which the first permanent bridges were built. For most of their lengths, these have remained important roads, and only a few of the original bridges remain.

Mr. J. J. Daniels, of Rockville, and the Kennedy family, of Rushville, are largely responsible for the great number of excellent timber bridges in the State, many of which are still in use. Mr. Daniels built about fifty

Note.—The paper by Robert Fletcher and J. P. Snow, Members, Am. Soc. C. E., was published in November, 1932, *Proceedings*. Discussion of this paper has appeared in *Proceedings*, as follows: January, 1933, by Henry B. Seaman, M. Am. Soc. C. E.; February, 1933, by Messrs. Jasper O. Draffin, E. K. Morse, and John W. Storrs; and March, 1933, by Messrs. C. J. Hogue, William A. Oliver, Philip G. Laurson, Richard S. Kirby, Wells N. Thompson, Benjamin Wilder Guppy, Ivan E. Houk, and William H. Baker.

[63] St. Paul, Ind.

[63a] Received by the Secretary January 30, 1933.

bridges, mostly in Parke and Vermillion Counties. He built a bridge over Sugar Creek, in Parke County, in 1861, which is still in use. This structure has a clear span of 200 ft and is the longest single-span covered bridge of record in Indiana. The Kennedy family built about sixty covered bridges in all, the first being at Dunlapsville, in Union County, in 1870, and the last near Fountain City, in 1918. Many of these bridges are still in use in Southeastern Indiana. The Burr truss bridge, at Rushville, built by the Kennedys in 1883, has two 125-ft spans with a 16-ft clear roadway, and is unusual in that it has an arcaded sidewalk on each side.

Two of the oldest timber trusses, still in service, are the Ramp and Raccoon Creek Bridges formerly on State Road No. 43, near Greencastle, which were built in 1837 and 1838. These bridges were constructed of local material, except for a few hand-made iron bolts and brace-pins. Both have two 11-ft roadways carried by three trusses, one of which separates the two roadways. The Ramp Creek Bridge has a clear span of 91.5 ft and the Raccoon Creek Bridge, one of 120 ft.

The Ramp Creek Bridge was removed in 1932 and re-erected over Salt Creek, at the north entrance to Brown County State Park, by the Indiana State Highway Commission. The arch ring, top chords, rafters, and roof framing are of yellow poplar; the posts, lower chords, and floor are of oak; while the floor-beams, joists, and sub-flooring are of walnut. Members that had decayed to any extent, were replaced by new timber. Oak was substituted for the damaged walnut. In all, 10 to 15% of the members were replaced. The abutments, which were carried to rock, are of concrete, faced with sandstone masonry. This timber bridge will have ample strength to carry the park traffic and should stand for many years as a monument to the early bridge engineers and their ingenuity in the use of local materials. Raccoon Creek Bridge is also to be replaced by a modern bridge to care for the heavy through traffic on State Road No. 43. Another timber bridge, maintained by the Indiana Department of Conservation, spans Sugar Creek, in Turkey Run State Park, at the Narrows Crossing. It has a span of about 100 ft.

R. P. Davis[64] and L. L. Jemison,[65] Members, Am. Soc. C. E. (by letter).[65a]—The authors of this paper merit the thanks of the Engineering Profession for their effort to record the history—from an engineering standpoint—of the fast-disappearing wooden bridge. Much of a general nature has been written on this subject in the last few years, most of which is, however, not from the engineer's point of view. The writers hope that, in order to round out this paper, more complete data, including general proportions, sizes of members, and details of framing, of the more historical types will be placed in this record.

During the latter part of the Eighteenth Century, in building highways to connect the Ohio country with the Eastern seaboard, the stone arch was widely used, as exemplified by those on the National Highway between Hagers-

[64] Dean, Coll. of Eng., West Virginia Univ., Morgantown, W. Va

[65] Bridge Engr., West Virginia State Road Comm., South Charleston, W. Va.

[65a] Received by the Secretary February 1, 1933.

town, Md., and Wheeling, W. Va. A little later the wooden bridge became the more popular, and from the beginning of the Nineteenth Century until after the Civil War wooden bridges were very common west of the Allegheny Mountains, the Burr type being almost exclusively used, without arches for the shorter spans. Nearly all these bridges were protected with roofs and side covering, with an open space at the top to permit proper ventilation.

Of particular historical value is the two-span combination arch and truss bridge over the Cheat River above Rowlesburg, W. Va. (see Fig. 35). This structure is on the old Northwestern Turnpike (later known as Federal Route 50), a highway built in 1832 under Col. Claudius Crozet, who had been an officer of artillery under Bonaparte in the Russian campaigns.

Completed in 1834 it has given excellent service for a period of more than 98 years. The material is white pine with all timbers hewn by hand; all bolts are of wrought iron with large heads. The bridge was built at a cost of \$18 000, or \$2.40 per sq ft of roadway, on the basis of a total length of 339 ft, and a clear roadway width of 22 ft. The arch ribs are composed of three 6 by $11\frac{1}{2}$-in. timbers, placed vertically one above the other, an arrangement not as common as placing them side by side horizontally. However, by this arrangement (see Fig. 35), less notching is required, and the full section of the arch may be secured. The stringers, 6 by 8 in. in section and spaced 3 ft 9 in. apart, are larger than those customarily found in highway bridges of the early days. The 7 by 10-in. floor-beams are hung from the lower chords by two 1-in. bolts. Why the more common and simpler method of placing these beams above the chords was not used, is not apparent. However, this method of support gave complete satisfaction, because the original bolts are still (1933) in an excellent state of preservation.

It is interesting to note that the tension web members of the trusses are not placed vertically, as is more usual, but inclined in a direction approximately normal to the arch ribs. Loads from the floor-beams first come to the trusses and are transferred to the arch rib by notching the so-called verticals and by placing wedges between the three rows of timbers forming the arch rib. It will be observed that these trusses have only a single system of diagonals. As these diagonals are framed so that they take only compression, the question naturally arises as to why a double system was not used. The sticks composing the arch rib and the lower chords are three panels in length. The type of splice used for the lower chord would be considered now a very poor one for a tension connection.

During recent years a double floor has been used on this bridge, together with longitudinal runways. These were necessary on account of the wide spacing of the stringers. When heavy truck loading was introduced, considerable weakness developed in the floor-beams in resisting horizontal shear near the ends. This weakening was due to the notching of the floor-beams at their ends and also to the cutting of holes to permit framing the diagonals of the lower lateral system.

Most of the timbers composing the trusses and the arches are in an excellent state of preservation, except the arch ribs near the springing points where, due to failure to carry the siding down in the early years, considerable

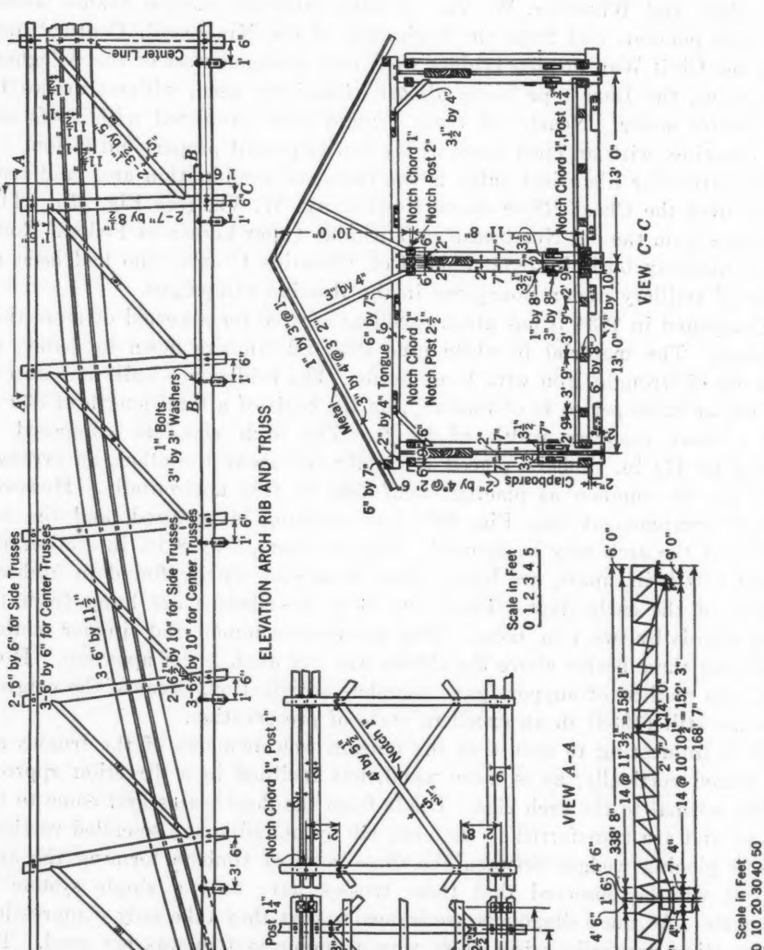

FIG. 35.—BRIDGE OVER CHEAT RIVER, NORTHWESTERN TURNPIKE, PRESTON COUNTY, WEST VIRGINIA

rotting developed. The bridge has also suffered under the action of some severe storms, as a consequence of which the trusses and arches are badly bowed out of line. The substructure is composed of stone masonry laid with close joints, without mortar. It is in good condition.

The bridge illustrated in Fig. 36 was on the famous James River and Kanawha Turnpike which was opened to traffic in 1800. The bridge was built in 1872 across the Greenbrier River, at Caldwell, W. Va. It had two spans, the arches being 228 ft long. Like the Cheat River Bridge it had a double roadway, the outside trusses being 25 ft, center to center. It was the second bridge to be built at the same location; the first one, constructed in 1807, was burned during the Civil War. The second bridge was built at a cost of \$12 500, or \$1.40 per sq ft of roadway, the superstructure resting on the original pier and abutments. When this masonry was torn down in 1932, the piers and abutments were found to be composed of stone masonry shells backed with small stones.

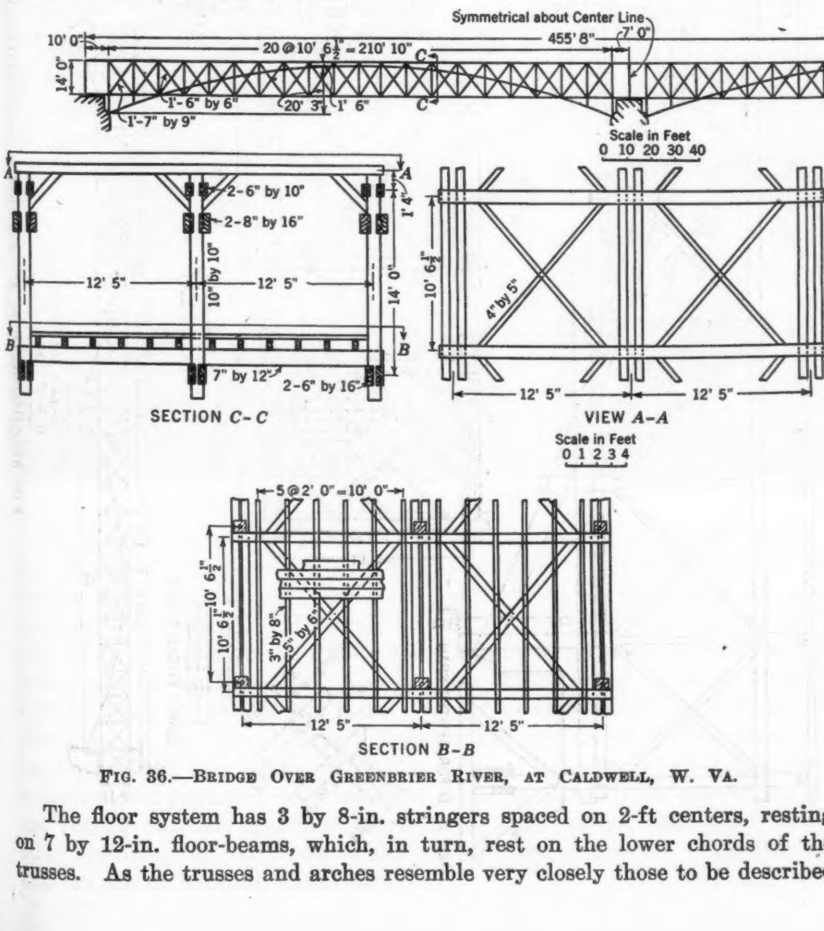

FIG. 36.—BRIDGE OVER GREENBRIER RIVER, AT CALDWELL, W. VA.

The floor system has 3 by 8-in. stringers spaced on 2-ft centers, resting on 7 by 12-in. floor-beams, which, in turn, rest on the lower chords of the trusses. As the trusses and arches resemble very closely those to be described

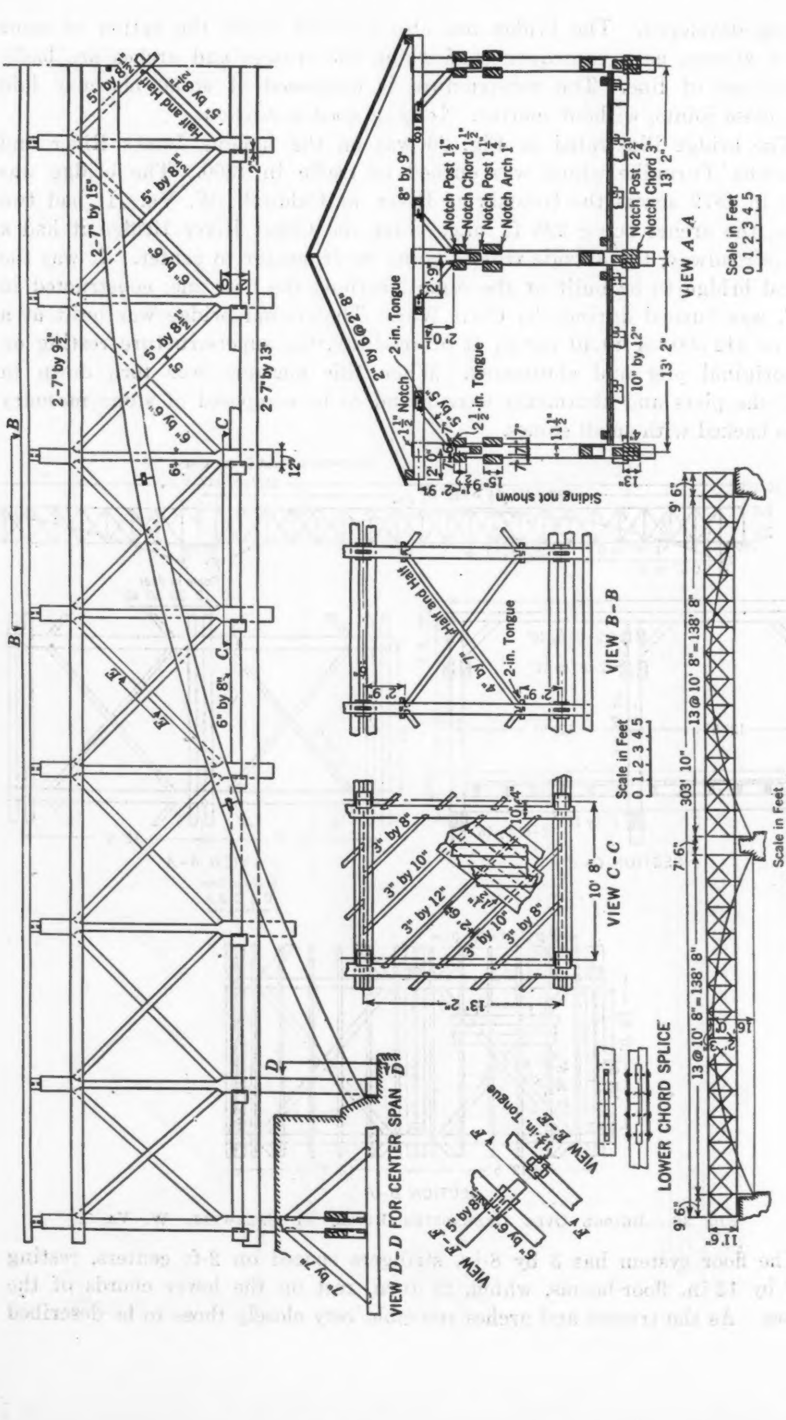

FIG. 37.—BRIDGE OVER TYGARTS RIVER, AT PHILIPPI, W. VA.

hereafter for the Philippi Bridge, neither these details nor the roof covering is shown on the drawing.

The bridge illustrated in Fig. 37 is over the Tygarts River, at Philippi, W. Va. Built in 1852, it is said to have been the scene of the first land battle of the Civil War. It has two spans, the arches being $138\frac{2}{3}$ ft long. The bridge has a peculiar stringer system composed of diagonal 3 by 8-in., 3 by 10-in., and 3 by 12-in. beams, notched half and half into the floor-beams and into 6 by 8-in. timbers placed on, and running parallel with, the lower chords. This type of connection has proved very weak under concentrated loading and has given considerable trouble by horizontal shearing at the ends of the stringers. No lower lateral system was used on this structure, probably for the reason that the stringers, being framed into the floor-beams and pinned as well, were expected to serve as bracing. The excellent type of fish-plate joint used for lower chord splicing should be noted.

FIG. 38.—INTERIOR VIEW OF PHILIPPI BRIDGE

The bridge was built by Lemuel Chenoweth, of Beverly, Va. (now W. Va.). Mr. Chenoweth was a bridge architect by profession, and it is said that his designs were original with him, and every principle was worked out with mathematical accuracy. He knew beforehand the shape and size of every piece of timber used in the framework of his bridges. The contract for this bridge was let at Richmond, Va. Bidders were present in large numbers with all kinds of models and plans. As far as appearances went, it is said that some of the New England Yankees had models of perfect form and beauty, painted and enameled in the highest art. Mr. Chenoweth's plain wooden model attracted little attention until he placed it on two chairs, one end resting on each, and then stood on his little bridge, and called on the other architects to put theirs to the test by doing the same. This feat got him the job of building the bridge.

Considerable decay of the arches has occurred near the springing lines, but all timbers that have been properly protected are in good condition. After eighty years of continuous use this structure still lines up almost perfectly.

The details of the trusses and arches are quite similar to those of the Waterford Bridge (Fig. 15) described by the authors. Fig. 38 gives a view of the interior of the Philippi Bridge.

The only metal in any of the bridges described was in the form of bolts, which were freely used to hold the various members, and their segments together; but in no case, except the floor-beam connections and the lower chord splices of the Cheat River Bridge, do these bolts take any primary stress. The transfer of load from one element to another was accomplished by direct bearing or by notching. Wedges were used quite freely to take care of shrinkage conditions and wooden pins were utilized in the bracing systems.

PETER T. LANDSEM,[66] ASSOC. M. AM. SOC. C. E. (by letter).[66a]—The authors deserve much credit for their thorough investigation of the development of wood bridges in the United States. However, the paper gives the impression of being an epitaph on wood bridges, which is not deserved. Certain developments in the more efficient use of lumber both in Europe and in the United States indicate that the world is probably on the threshold of a renaissance in wooden bridge construction.

In general, information on the period from the collapse of the Roman Empire up to 1 500 A. D., seems to be lacking in the authors' treatise. Improvements were small, which is astonishing in view of the highly developed church architecture, both Romanesque and Gothic. Most bridges were simple trestles. An outstanding example was the one built over the Rhine, at Basel, Switzerland, in 1225, and first replaced in 1903.

It might be of interest to state that Palladio did not consider himself the inventor of the truss. He refers to a truss with parallel chords, verticals, and single compression diagonals found in use in Germany by Picheroni de Mirandola.

The authors state that "covered or roofed-in bridges were distinctly not a feature in European practice," yet there is evidence that covered bridges were known in Germany and Switzerland long before the famous Ulrich Grubenmann and Josef Ritter became active in bridge construction early in the Eighteenth Century. One of the oldest of such bridges from the Middle Ages, still in existence, is the picturesque "Kapellbrücke" over the Reuss, in Luzerne, Switzerland, built in 1333. Similar bridges were built in the Sixteenth Century with spans up to 22 m. In 1658, a covered bridge with a span of 32 m was built over the Rhine, at Stein-Säckingen, Germany. Dr.-Ing. K. Schaechterle,[67] has called attention to a number of covered bridges, and advice concerning the construction of covered bridges has been published by L. Chr. Sturm;[68] J. Leupold;[69] Johan Wilhelm;[70]; and Johann Vogel.[71]

[66] Asst. Constr. Engr., National Committee on Wood Utilization, U. S. Dept. of Commerce, Washington, D. C.

[66a] Received by the Secretary February 4, 1933.

[67] "Holzbrücken," 1927.

[68] "Architectura civili-militaris," 1719.

[69] "Theatrum pontificiale," 1726.

[70] "Architectura civilis," 1682.

[71] "Die Moderne Baukunst," 1708.

Developments in Europe in the Nineteenth Century.—The authors of this treatise do not trace the development of wooden bridges in Europe after the middle of the Eighteenth Century, and do not cite any examples of such bridges built in the latter part of the Nineteenth Century in the United States.

A brief summary of the development in Europe may be of interest. About 1800, Funk, in Germany, applied to bridges a wooden arch that had been used for roofs by the French architect, de l'Orme, in the latter part of the Sixteenth Century. The arches were made of boards on edge, cut to shape, and nailed and doweled together. This system was not satisfactory, however, because in shaping the boards the grain of the wood was cut too much and because water entered so easily into the vertical joints.

In 1809, Wiebeking, another German, used for bridges the system generally attributed to the French Colonel Emy, although not in use by him until 1819. The boards were laid flat and nailed and bolted together. This type of arch, however, was too flexible even when braced. Load tests on both de l'Orme's and Emy's systems were carried out by the French military engineer, P. Ardant.[72]

Although the railroads increased the demand for bridges early in the Nineteenth Century, no significant improvements originated in Europe. It was left to the United States to develop the combined arch truss into the framed trusses of Long, Howe, and Pratt. From the middle of the century the art of wooden bridge building went into a decline. This must be attributed mainly to lack of knowledge concerning the properties of wood and concerning reliable framing methods. Engineers of that day lost interest in wood construction. In fact, the only new idea that has come to the writer's attention are the gallow bridges in Trondhjem, Norway.

Modern Developments in Wood Construction.—New interest in wood construction was created about 1900 by carpenter-contractors who, having acquired some technical knowledge, put up structures that caught the interest of engineers. The engineers have carried the developments further. Therefore, in Germany to-day, they speak about "Ingenieur-Holzbau," or engineering wood-construction.

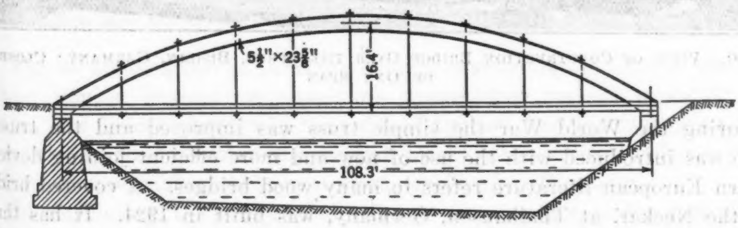

FIG. 39.—SECTION OF FOOTBRIDGE OVER THE WIESE RIVER AT BASEL, SWITZERLAND

[72] "Theoretischpraktische Abhandlung über Anordnung und Konstruktion der Sprengwerke von grosser Spannweite," von A. v. Kaven, 1847.

Stephan made Emy's arch stiffer by building it like a truss of curved laminated chords, dowled together, and braced by cross-diagonals. This system, however, was used only for roof trusses. The railway station in Copenhagen, Denmark, is a good example.

Otto Hetzer introduced his laminated glued-wood arch in 1907. It consists of boards laid flat, curved, and glued together, the glue being water-resistant and having at least the same strength as the wood. An early example is the foot-bridge over the Wiese, at Basel, Switzerland (see Fig. 39), which was built in 1910. The roadway is suspended from the two-hinged arches, 33 m (108.3 ft) in span. It was computed for a load of 350 kg per sq m (72 lb per sq ft) and cost 6 200 francs, which was only one-half the nearest bid in steel. Th. Gesteschi[73] and H. Lewe[74] mention other arch bridges.

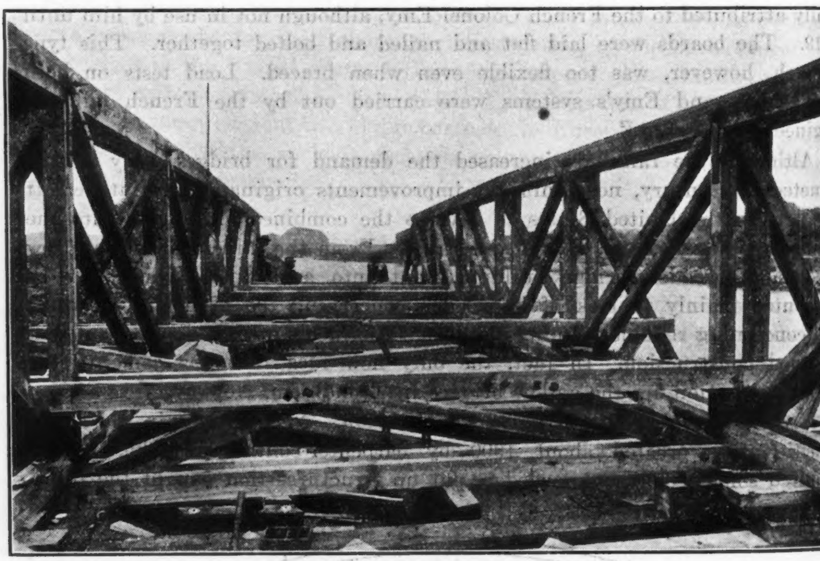

FIG. 40.—VIEW OF CONSTRUCTION BRIDGE OVER THE SPREE, BERLIN, GERMANY; CLOSE-UP OF ONE SPAN

During the World War the simple truss was improved and the trussed frame was introduced with the use of new and more efficient joining devices. Modern European literature refers to many wood bridges. A covered bridge over the Neckar, at Thalhausen, Germany, was built in 1924. It has three spans, 62.3 ft long and two spans, 32.8 ft long. Others are: A suspension bridge at Sulitjelma, Norway, 266 ft long (maximum span, 177 ft); a pipe line suspension bridge in Finland, with four spans of 167 ft each and a total length of 984 ft; a footbridge at Neumünster Railway Station, in Germany (maximum single spans, 131 ft, and total length, 541 ft); and a military

[73] "Der Holzbau."

[74] "Neuere Brückenbauten in Holz," *Bauingenieur*, 1921, Vol. 17.

road bridge in Norway (span, 85 ft). Well known foreign sources of information are: Th. Gesteschi;[73] C. Kersten;[75] and A. Laskus.[76]

Introduction of Modern Wood Construction Methods in the United States.—It has been recognized that the chief drawback in the use of wood to-day in bridge construction has been the inefficient joint. In bolted connections the safe loads determined by the various handbook methods differed widely. This situation, fortunately, has been relieved by an extensive series of tests[77] carried on for years at the Forest Products Laboratory, U. S. Department of Agriculture. The Engineering Staff of the National Committee on Wood Utilization, U. S. Department of Commerce, in co-operation with the Forest Products Laboratory, U. S. Department of Agriculture, has been investigating still better joining devices which, as tests show, may increase the load capacity from two to six times (and even more) than that of an ordinary bolted joint.

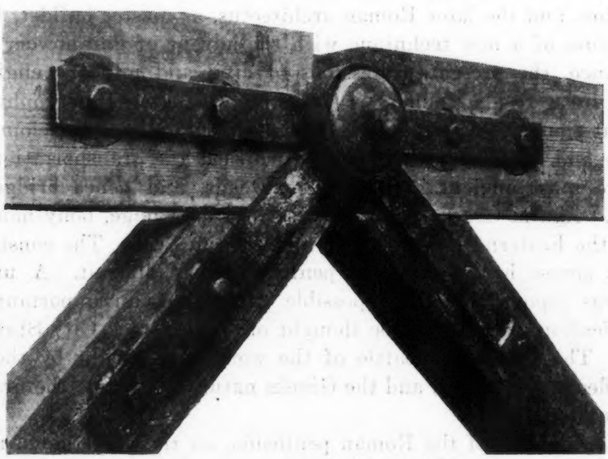

FIG. 41.—VIEW OF JOINT COMPOSED OF MODERN CONNECTORS

Fig. 40 shows a bridge in which modern connectors are used in the joints. These joining devices or connectors, comprising various types of rings, plates, and disks embedded in the sides of timbers (see Fig. 40), are used for transmitting the load from one member to the other. Fig. 41[78] is the view of a joint composed of modern connectors developed in Germany.

In concluding, the writer would like to call attention to the fact that the State of Oregon is building timber highway bridges, using the money thus saved for better road surfaces.

[75] "Freitragende Holzbauten."

[76] "Hölzerne Brücken."

[77] *Bulletin No. 332*, "The Bearing Strength of Wood Under Bolts," by George W. Trayer, 1932.

[78] "Modern Connectors for Timber Construction," by Nelson S. Perkins, Peter T. Landsem, and George W. Trayer, *Bulletin No. 24*, National Committee on Wood Utilization, U. S. Dept. of Commerce.

J. K. FINCH,[79] M. AM. SOC. C. E. (by letter).[79a]—One or two books on American timber bridges have appeared in recent years, but this paper is the first to record and discuss some of the interesting technical details involved in early timber bridge construction. While the men who built these bridges were simply self-taught Yankee carpenters, their abilities were of the first order and from their labors the truss bridge, a typically American contribution to engineering progress, evolved.

The writer has no details to add to the record of these master builders as given by Messrs. Fletcher and Snow. There are, however, a few notes that might be of interest on the place this timber-bridge era occupies in the great epic of engineering achievement.

The authors mentioned the lack of bridges in ancient Greece. The same observation would hold for Egypt and Mesopotamia. This was not due to any lack of ability on the part of the Greek architecton, or master builder. (The name, architect, is derived from this title but it is preferable to translate architecton, and the later Roman architectus, as master builder, for, until the introduction of a new technique with the advent of gun-powder in the early Renaissance, the present professions of civil and military engineering and architecture were all included in this single name). Topography was a most important element in shaping the course of engineering development. Thus, the Nile and the Tigris-Euphrates furnished the transportation routes for these two most ancient civilizations. Roads, and hence bridges, were not required. Again, Greece has been described as a huge, bony hand stretching out into the Eastern Mediterranean and Aegean Seas. The construction even of trails across her rock-ribbed peninsulas was difficult. A unified Greek nation was topographically impossible. Roads, as an important element of communication, were not to be thought of. The Greek City States turned to the sea. The first naval battle of the world was fought by the Greek and Persian fleets at Salamis, and the Greeks naturally became the first of ancient harbor builders.

The topography of the Roman peninsula, on the other hand, made it physically possible to consolidate this great area and its many tribes into a single nation from the Alps to the heel of Calabria. This opportunity was taken advantage of by the strongest tribe, the Latin race, and the construction of roads, and necessarily of bridges, became an inevitable, a basic element, to this consolidation. The Alps shut off France from Italy, but France also was consolidated by another great Roman road and bridge-building development, while communication between the seaports of Southern France and Northern Italy furnished the connecting link.

Centuries later, in the United States, economic pressure from the seaboard States, the struggle of each State to secure trade advantages from the expansion westward which followed the Revolution, combined with the topographic problems involved, made the American canal and railroad era, and its accompanying bridge development, inevitable. It was not only Yankee ingenuity, or the force and initiative of a pioneer people, that made the United States

[79] Renwick Prof. of Civ. Eng., Columbia Univ., New York, N. Y.

[79a] Received by the Secretary February 18, 1933.

the greatest nation of bridge builders the world has ever known. Topography and economic pressure have forced the people of this country to be ingenious and progressive in this particular field.

It is also interesting to reflect on some other factors and forces that led American engineers to develop the truss bridge.

Palladio's famous timber truss of about 1540, mentioned by the authors, is well known. The first metal truss, however, had been built centuries earlier, under the Roman Engineer-Emperor Hadrian, to support the portico of the Pantheon at Rome. Undoubtedly, the truss form was used by the Greeks and, later, by the Romans in many roof constructions. Timber, however, was recognized as temporary, and the fire hazard was also considered. Hadrian's truss, for example, was of bronze and was clearly a pioneer attempt at fireproof building construction. (It was removed and melted for making cannon under Pope Urban VIII in 1625, but early drawings show its form and details.) Timber bridges, although undoubtedly used, were replaced by fireproof, durable stone arches whenever economic conditions warranted permanent construction. Hadrian's effort to build framing in metal was interesting, but of no economic importance because metal was too costly. In fact, the truss, an invention in framing, did not emerge, was not discovered or invented, until a civilization arose which was based almost entirely on timber construction. Colonial America was such a civilization.

David Stevenson, the British engineer, writing[80] in 1838 defended the use of timber and temporary constructions by American engineers, a practice which had been criticized by European workers. He pointed out that labor was scarce in the States, funds were not available for costly stone constructions, and the unknown future requirements as to loads and sizes of structures made temporary construction advisable. Construction in timber, therefore, represented not only economical design, but also good engineering and business foresight.

How much such pioneers in this field as Palmer, Burr, and Wernwag were indebted to earlier European bridge builders for their basic ideas, will probably never be definitely known. "Copying" may not have occurred at all. Indeed, books were scarce, and American Colonial carpenters would not be expected to have or see them. This point, however, is unimportant. These men and their associates, as well as earlier workers, apparently had no very clear conceptions of truss action, or at least did not have full confidence in such framing. Their bridges were "statically indeterminate" combinations or arches with "stiffening" trusses. Nevertheless, the germ of the truss principle existed in their work, and ultimately the truss emerged from their composite structures as a form capable in itself of carrying the full load. Town, Long, and, finally, Howe (1840) saw this element in the work of Palmer, Burr, and Wernwag, and the truss was thus brought forward as a distinct and separate structural form.

During this period of truss evolution, British engineers stuck stubbornly to their metal arches, girders, and "tubular" bridges. American engineers,

[80] "Sketch of the Civil Engineering of North America," pub. by John Weale, Lond., 1838.

following Haupt and Whipple, began to analyze the stresses in trusses. There was an era of patented trusses in which Pratt, Warren, Bollman, and others contributed to the evolution of economic truss forms. The timber and wrought iron was translated into the cast and wrought-iron construction of the Sixties, and then into the all-steel truss of the Seventies and Eighties. Finally, the engineers of the present day have contributed the great alloy steel spans of the modern highway era.

The work of the early American timber bridge builders, therefore, represents only one step in the history of a great development. Their contribution to engineering progress, however, entitles these pioneer American bridge engineers to a prominent place in the annals of a great profession.

A. A. EREMIN,[81] ASSOC. M. AM. SOC. C. E. (by letter).[81a]—A number of ancient wooden bridges have been described (in Russian) by Mr. L. Nikolai,[82] a reference to which should be made in connection with this paper. For example, he shows the Trajan Bridge over the Danube River at the Iron Gates built by Apollodorus about 109 A. D. This is the bridge to which reference was made by the authors. The clear span was 118 ft, and the stone piers were carried 20 ft below water level. The Trajan Bridge is known as the first wooden arch bridge.

A freedom of the creative imagination at the beginning of the Nineteenth Century is well illustrated by Mr. Burr's bridge,[31] at Schenectady, N. Y., built in 1808. The curved chords were formed of eight 4 by 14-in. planks, spiked, and bolted together. After twenty years of service the bridge was reinforced with additional intermediate piers built in the middle of each span, and, in 1873, the structure was replaced by a metal bridge.

Among the notable wooden bridges built in Russia, Mr. Nikolai has described the "rainbow" arch bridge with suspended deck and a clear opening of 257 ft, spanning the Wepr River at Ivan Gorod Fortress.

In crossing navigable rivers the Russian builders used a removable bridge with draw-spans that were easily opened. The bridge was removed in winter when the river was covered with ice, and in spring, to permit a heavy mass of floating ice to pass at high water.

A primitive type of removable bridge is the floating type of structure. The deck of a floating bridge built in Riga (Fig. 42), was formed of planks

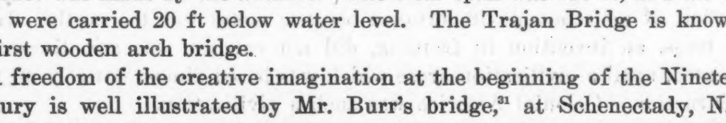

FIG. 42.—FLOATING BRIDGE

3 in. thick on 6 by 6-in. joists spaced at 3-ft centers and placed on 6 by 12-in. beams spaced at $3\frac{1}{2}$-ft centers. The beams were bolted to the 13 by 13-in. floating longitudinal beams. The average life of this type of floating bridge was about ten years. Later, the floating bridges were replaced with pontoon bridges, the spans being carried on wooden or metal barges.

[81] Associate Bridge Designing Engr., State Highway Comm., Sacramento, Calif.
[81a] Received by the Secretary February 25, 1933.
[82] "Bridges," by L. Nikolai, Russian Edition, 1901.
[31] *Transactions*, Am. Soc. C. E., Vol. XXI (July, 1889), Pl. VIII.

AKSEL ANDERSEN,[83] M. AM. SOC. C. E. (by letter).[83a]—The following data on wooden railroad bridges built in Norway between 1860 and 1875 is of interest in connection with this instructive paper on the development of wooden bridges. During that period the Norwegian Government constructed several important railway lines, including the northern half of the railroad between Oslo and Trondhjem, and several lines in the southeastern part of the country.

A great variety of conditions were encountered, involving the bridging of numerous rivers and deep valleys in mountainous country, with poor transportation facilities. Skilled carpenters and excellent pine timber were available, however, at low cost, and the selection of wood as the principal building material was natural. Stone or iron bridges would have been much more expensive and would have required a longer construction period.

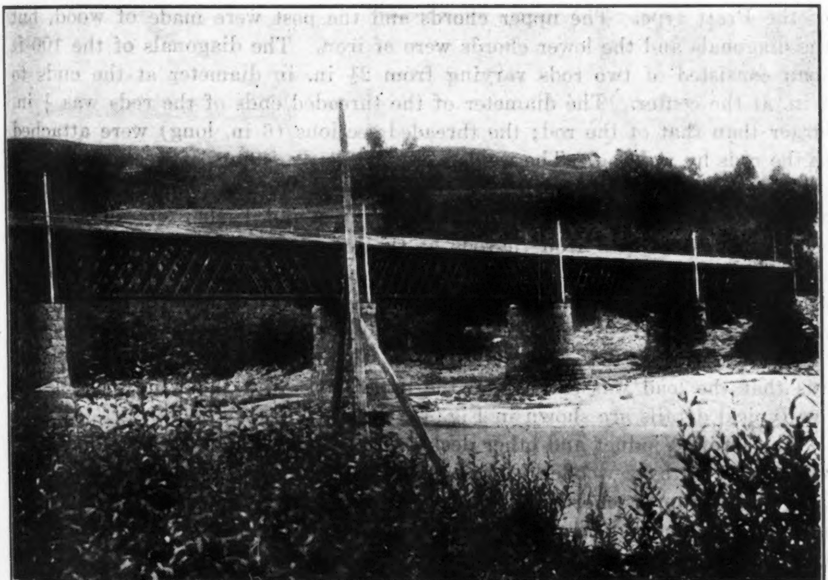

FIG. 43.—VIEW OF GULFOSS BRIDGE

In the lowlands the pile trestle was a common type, with spans as great as 40 ft. Compound keyed stringers with queen or king-braces were used for the longer spans. Practically all structures of this type were built of round, or half-round timber, and the joints were held together with iron screw-bolts.

For river crossings and high viaducts, with spans up to 100 ft, Pratt trusses and Howe trusses, with one and, in a few cases, with two systems of web members were used. Fig. 43 shows a bridge (Gulfoss Bridge), with four continuous-truss spans of 100 ft and several shorter spans. The continuous trusses are of the original Howe type with two web systems, very similar to

[83] Forest Hills, N. Y.

[83a] Received by the Secretary March 9, 1933.

the trusses of the bridge over the Connecticut River, at Springfield, Mass., shown in Fig. 24. The only apparent difference is the absence of braces against the piers below the lower chord. The same system of continuous 100-ft spans, with two systems of web members, was used in two other railroad bridges, and one of them (the bridge over the Gula River, at Stören), is still in use (1933) after nearly seventy years of service. (In this connection note the statement by Messrs. Fletcher and Snow under "The Howe Truss," that "it is not known to the writers that there has been another bridge built of the Howe type with webs of two systems)." The Gulfoss Bridge and several other through railroad bridges were protected by roofs, extending about 6 ft outside the trusses, with no side coverings.

Deck spans with simple Howe trusses, or Pratt trusses, supported by timber piers on masonry bases, were used in a number of high viaducts. The Dröja Viaduct, shown in Fig. 44, had one 100-ft, and even 45-ft, truss spans of the Pratt type. The upper chords and the post were made of wood, but the diagonals and the lower chords were of iron. The diagonals of the 100-ft span consisted of two rods varying from $2\frac{1}{8}$ in. in diameter at the ends to 1 in. at the center. The diameter of the threaded ends of the rods was $\frac{1}{4}$ in. larger than that of the rod; the threaded sections (6 in. long) were attached to the rods by welding. The welds were, obviously, points of weakness.

The wind system consisted of timber crosses in the plane of the upper chords with iron tie-rods at the panel points, and timber cross-frames at each panel point with iron tie-rods at the lower chords. The writer recalls an incident, in about 1916 or 1917, that proved the efficiency of this type of wind stiffening. One of the iron diagonals in a Pratt truss of a high timber viaduct failed, and several trains had probably passed over the viaduct before the break was detected. The only reason why the structure did not collapse was that the load was carried by the other truss and the wind system. A few typical details are shown in Fig. 44.

The Dröja Viaduct and other deck bridges and viaducts of the same type, were protected by a wooden roofing, extending about 3 ft beyond the outside faces of the upper chords. The trough-shaped part between the rails was made of longitudinal boards, covered with iron plates in order to reduce the fire hazard. The roofing outside the rails consisted of two layers of 1-in. by 6-in., transverse boards sloping slightly outward. The boards in each layer were placed with a spacing of 2 in., thus overlapping the boards of the other layer by 2 in. The contact surfaces between the two layers were provided with grooves for drainage (see Fig. 44(*d*)). A similar system was used for wooden floors of highway bridges. The wearing surface consisted of longitudinal planks, and the loads were carried to the stringers by two layers of overlapping and grooved planks.

Simple Howe trusses, with iron rods as verticals, were used in a number of railroad bridges and viaducts. In some of them, the angle blocks were made of wood (usually oak), but more frequently of cast iron.

Practically all wooden bridges designed and built by the Norwegian Government Railways in the aforementioned period have been replaced, in recent years, by stone or steel bridges, or have been eliminated by re-location

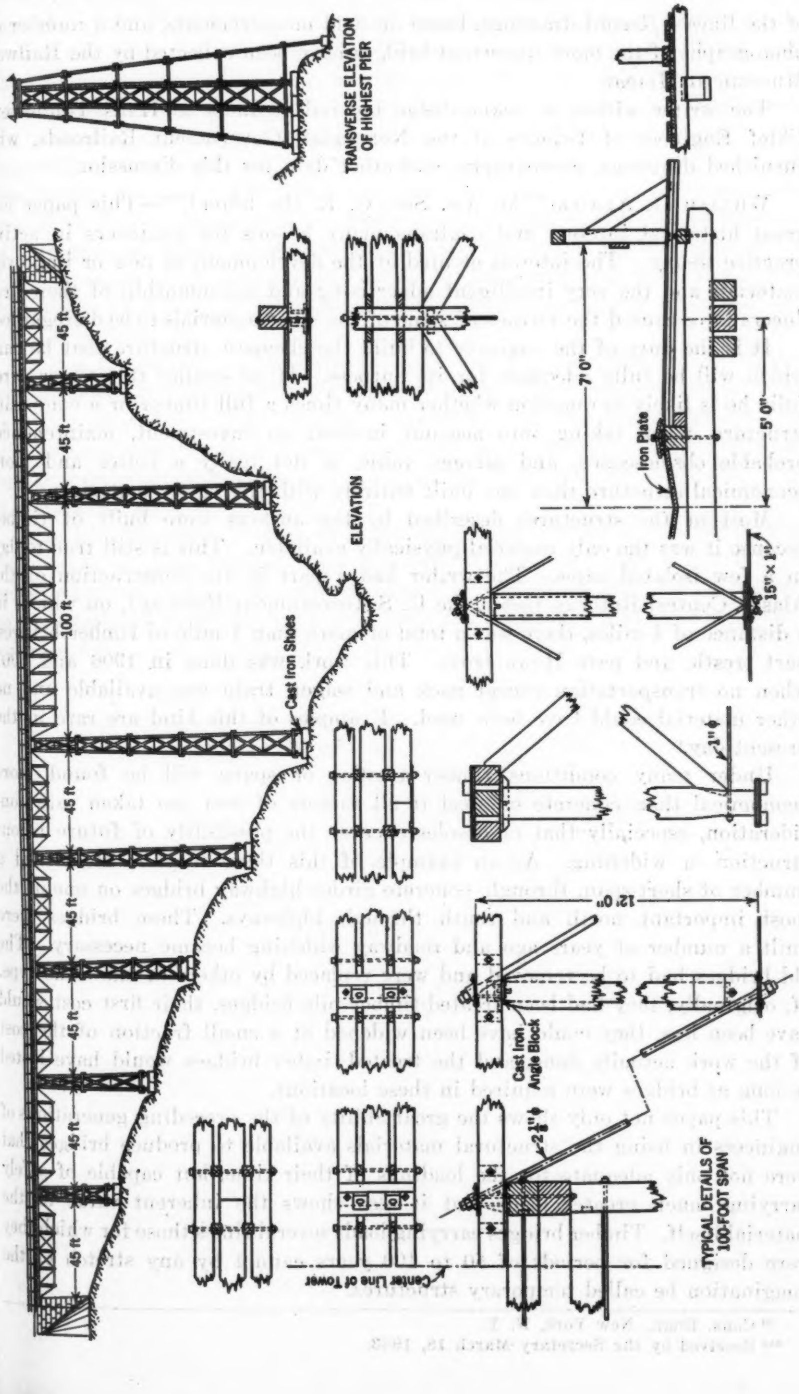

FIG. 44.—TYPICAL DETAILS OF TRUSS SPANS, DROJA VIADUCT

of the lines. Record drawings, based on field measurements, and a number of photographs of the more important bridges have been collected by the Railway Museum at Hamar.

The writer wishes to acknowledge his indebtedness to Hans Tönnessen, Chief Engineer of Bridges of the Norwegian Government Railroads, who furnished drawings, photographs, and other data for this discussion.

WILLIAM G. ATWOOD,[84] M. AM. SOC. C. E. (by letter).[84a]—This paper has great historical interest and contains many lessons for engineers in active practice to-day. The interest created by the development of new or improved materials and the very intelligent advertising and salesmanship of their producers have caused the virtues of some of the older materials to be disregarded.

It is the duty of the engineer to build the cheapest structure that he can, which will be fully adequate for its purpose. If he studies this paper carefully he is likely to question whether many times a full timber or a composite structure, when taking into account interest on investment, maintenance, probable obsolescence, and salvage value, is not really a better and more economical structure than one built entirely with the newer materials.

Most of the structures described by the authors were built of timber because it was the only material physically available. This is still true to-day in a few isolated cases. The writer had a part in the construction of the Alaska Central Railway (later, the U. S. Government Railway), on which, in a distance of 4 miles, there was a total of more than 1 mile of timber bridges, part trestle and part Howe truss. This work was done in 1906 and 1907 when no transportation except pack and wagon train was available and no other material could have been used. Examples of this kind are rare in the present day.

Under many conditions timber trestles or spans will be found more economical than concrete or steel if all factors of cost are taken into consideration, especially that of obsolescence or the possibility of future reconstruction or widening. As an example of this there may be mentioned a number of short-span, through, concrete girder highway bridges on one of the most important north and south through highways. These bridges were built a number of years ago and roadway widening became necessary. The old bridges had to be removed and were replaced by others of the same type. If, originally, they had been treated-timber pile bridges, their first cost would have been less, they could have been widened at a small fraction of the cost of the work actually done, and the treated-timber bridges would have lasted as long as bridges were required in these locations.

This paper not only shows the great ability of the preceding generations of engineers in using the structural materials available to produce bridges that were not only adequate for the loadings of their time, but capable of safely carrying much greater loads, but it also shows the inherent value of the material itself. Timber bridges carrying loads several times those for which they were designed for periods of 50 to 100 years cannot by any stretch of the imagination be called temporary structures.

[84] Cons. Engr., New York, N. Y.

[84a] Received by the Secretary March 18, 1933.

When giving credit to their professional predecessors engineers should remember that they did not have the information that is available at the present day even regarding such an old structural material as timber; nor was treated timber available in any quantity until the last fifty years. As a result of the work of the U. S. Forest Products Laboratory, the Department of Commerce, and the various associations of timber producers and timber treaters, together with the engineering societies, there is available for the present-day designer information that permits him to select his timber and treatment with a certainty as to strength and durability that is not exceeded in the case of other structural materials. Engineers of this generation, therefore, may build stronger and more durable structures with less timber than their predecessors. The paper under discussion shows what the past generation could do.

It is a fact which will probably be questioned by few who are conversant with present practices, that frequently advantage is not taken of the information now available. The Highway Department of one important State is now using timber grading rules and specifications written thirty years ago, and an important city is using creosoting specifications nearly as old. This is a waste of money.

Men like Timothy Palmer whose bridges are described in the paper, were in advance of their time. This generation of engineers may well ask themselves whether in their frequent disregard of the oldest and one of the best of the available structural materials, they are worthily "carrying the torch of progress."

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852

DISCUSSIONS

DISTRIBUTION OF SHEAR IN WELDED CONNECTIONS

Discussion

By Messrs. P. Wilhelm Werner, and Isamu Ohno

P. Wilhelm Werner,[16] Assoc. M. Am. Soc. C. E. (by letter).[16a]—A rather important problem is treated in this paper. The author has shown that the shear is concentrated near the ends of the weld. It may be of value to know whether there are any means by which a more even distribution of the shear, and thus a better utilization of the length of the weld, could be effected.

Consider the more general case represented by Fig. 12, showing diagrammatically a weld of the same type as that referred to by Mr. Troelsch, but in which the area of the connecting bars decreases uniformly from the full section to a fraction thereof at the ends of the weld. For the sake of simplicity in the following analysis it may be assumed that the two bars are equal, so

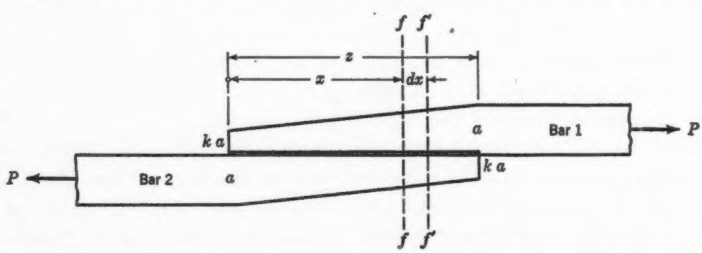

Fig. 12.—Diagrammatical Representation of Weld with Tapering Ends of Bars

that the full section is $a_1 = a_2 = a$, and that the area of the end sections is ka, in which, $k < 1$. The author's assumptions as regards the distribution of the direct stress in the bars, etc. (which may be considered to hold true for the type and relative sizes of the bars in question), have been adopted.

Note.—The paper by Henry W. Troelsch, M. Am. Soc. C. E., was published in November, 1932, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: February, 1933, by Messrs. F. T. Llewellyn, A. S. Woodle, Jr., Milton Male, William Hovgaard, Charles W. Chassaing, and F. E. Fahy; and March, 1933, by W. H. Jameson, Assoc. M. Am. Soc. C. E.

[16] Stockholm, Sweden.

[16a] Received by the Secretary January 26, 1933.

The writer has found it more convenient to take the end of Bar 1 as the origin of co-ordinates. Otherwise, using the same nomenclature as the author, the equilibrium condition for that part of Bar 1 on one side of Section *f-f*, may be written, as follows:

$$s_2 = \frac{s_0 - s_1 \left[k + \frac{x}{z} (1 - k) \right]}{1 - \frac{x}{z} (1 - k)} \quad \text{..................(33)}$$

in which, $s_0 = \frac{P}{a}$.

From the equilibrium of horizontal forces on the element between Sections *f-f* and *f'-f'*:

$$v = \frac{a}{z\,N} \left\{ s_1 (1 - k) + \frac{d s_1}{dx} \left[z\,k + x (1 - k) \right] \right\} \quad \text{........(34)}$$

from which,

$$q = \frac{v}{D} = \frac{a}{z\,N\,D} \left\{ s_1 (1 - k) + \frac{d s_1}{dx} \left[z\,k + x (1 - k) \right] \right\} \quad \text{.....(35)}$$

and by differentiating,

$$\frac{dq}{dx} = \frac{a}{z\,N\,D} \left\{ \left[zk + x (1 - k) \right] \frac{d^2 s_1}{dx^2} + 2 (1 - k) \frac{d s_1}{dx} \right\} \quad \text{.....(36)}$$

According to the author's Equation (5), combined with Equation (33),

$$\frac{dq}{dx} = \frac{s_1 - s_2}{E} = \frac{z}{E} \, \frac{s_1 (1 + k) - s_0}{z - x (1 - k)} \quad \text{................(37)}$$

Equating the values of $\frac{dq}{dx}$ in Equations (36) and (37), substituting $c = \frac{z^2 N D}{a E}$, and reducing,

$$\frac{d^2 s_1}{dx^2} + \frac{2 (1 - k)}{z\,k + x (1 - k)} \, \frac{d s_1}{d\,x} = \frac{c \left[s_1 (1 + k) - s_0 \right]}{\left[z\,k + x (1 - k) \right] \left[z - x (1 - k) \right]} \quad \text{...(38)}$$

which corresponds to the author's Equation (7), with $a_1 = a_2 = a$.

As far as the writer knows, it is not possible to integrate Equation (38) directly. A numerical solution is obtained, however, by breaking up the equation into two simultaneous differential equations of the first order. This will be illustrated by the following examples.

Example 1.—Letting $k = \frac{1}{2}$, and assuming the same numerical values as in the author's Example 2, $c = \frac{6^2 \times 4 \times 15\,000}{4 \times 30\,000} = 18$ in., and $s_0 = \frac{64}{4} = 16$ kips per sq in. After reducing, Equation (38) becomes,

$$\frac{d^2 s_1}{dx^2} + \frac{2}{6 + x} \, \frac{d s_1}{dx} = \frac{108 \left(s_1 - \frac{2}{3} s_0 \right)}{(6 + x)(12 - x)} \quad \text{..............(39)}$$

In Equation (39), substituting $\frac{ds_1}{dx} = w$ and $\frac{d^2 s_1}{dx^2} = \frac{dw}{dx}$:

$$\frac{dw}{dx} = \frac{108\left(s_1 - \frac{2}{3}\, s_o\right)}{(6 + x)\,(12 - x)} - \frac{2\,w}{6 + x} \quad \text{..............} (40a)$$

and,

$$\frac{ds_1}{dx} = w \quad \text{..........................} (40b)$$

Equations (40*a*) and (40*b*) are two simultaneous differential equations of the first order, which can be solved by the numerical integration method developed by Runge.[17] To save space, the somewhat elaborate numerical calculations will be omitted here, and only the result will be given. It may be mentioned that the values of both s_1 and $w = \frac{ds_1}{dx}$ are obtained simultane-

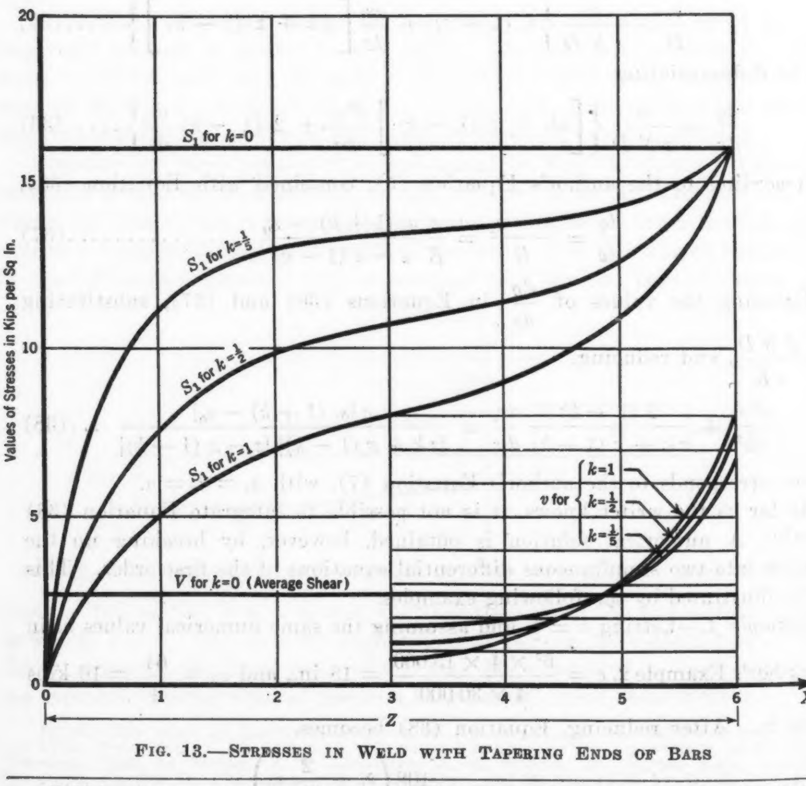

FIG. 13.—STRESSES IN WELD WITH TAPERING ENDS OF BARS

[17] "Ueber die numerische Auflösung von Differentialgleichungen", von C. Runge, *Mathematischen Annalen*, Bd. 46 (1895), or "Leitfaden sum graphischen Rechnen", von R. Mehmke, Leipzig, 1917.

ously in the same integrating operation. The values of s_2 and v are then calculated according to Equations (33) and (34), respectively, with $k = \frac{1}{2}$.

The result is shown graphically in Fig. 13. It is seen that the maximum shear at the ends of the weld has decreased only a comparatively moderate amount, namely, from 8.04 to about 7.5 kips per sq in.

As the shear is symmetrical about the center line of the weld, it has been plotted for only one-half the length of the weld. The unit stress, s_2, is identical to s_1, but reversed, and, therefore, has not been given in Fig. 13.

Example 2.—Let $k = \frac{1}{5}$, and the other values as given previously. Equation (38) then becomes:

$$\frac{d^2 s_1}{dx^2} + \frac{4}{3 + 2x} \frac{ds_1}{dx} = \frac{135 \left(s_1 - \frac{5}{6} s_0 \right)}{(3 + 2x)(15 - 2x)} \qquad (41)$$

from which,

$$\frac{dw}{dx} = \frac{135 \left(s_1 - \frac{5}{6} s_0 \right)}{(3 + 2x)(15 - 2x)} - \frac{4w}{3 + 2x} \qquad (42a)$$

and,

$$\frac{ds_1}{dx} = w \qquad (42b)$$

The integrals of Equations (42*a*) and (42*b*) are obtained in the same manner as for Equations (40*a*) and (40*b*). The values of s_2 and v are then calculated according to Equations (33) and (34), respectively, with $k = \frac{1}{5}$.

This result is shown also graphically in Fig. 13. It is seen that in this case the maximum shear at the ends of the weld has further decreased, that is, to about 6.5 kips per sq in. This decrease is considerable when compared with the average shear, which is $v = 2.667$ kips per sq in.

For comparison, the values of s_1, s_2, and v from the author's case, Example 2, have also been plotted in Fig. 13. It appears that the more the bars are tapered, the more evenly will the shearing stresses be distributed over the length of the weld. If the ends of the bars were wedge-shaped ($k = 0$), the shear would be constant throughout the length of the weld, and would be equal to the average shear. The unit stresses in the bars would then likewise be constant throughout the length of the weld, and would be equal to s_0.

Even a short taper of the very ends of the bars, reaching over only part of the weld, would be likely to effect a considerable decrease of the shear at the ends of the weld. A reduction of the shearing stresses in these places is desirable not only because it effects a more favorable distribution of the stresses, but possibly also because a concentration of the stresses at points of discontinuation, such as those constituted by the ends of the weld, may encourage the beginning of failure. As far as the writer is aware, a taper reaching at least over part of the weld at the very ends of the bars should not generally meet with any practical difficulties.

Isamu Ohno,[18] Esq. (by letter).[18a]—In connection with the paper by Mr. Troelsch the writer wishes to call attention to the application of the method to a plate to which a shorter reinforcing plate has been welded. Let Fig. 14 be a representative part of such a reinforced plate, for which the center of co-ordinates is at Point O (Fig. 14 (d)). Since the reinforcing plate is sym-

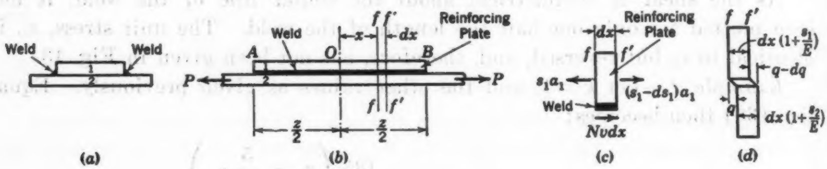

Fig. 14

metrical, only the right half, $O\,B$, need be considered. Using the author's notation and logic, his formulas, Equations (2), (8), (9), and (10) are derived as in the original paper.

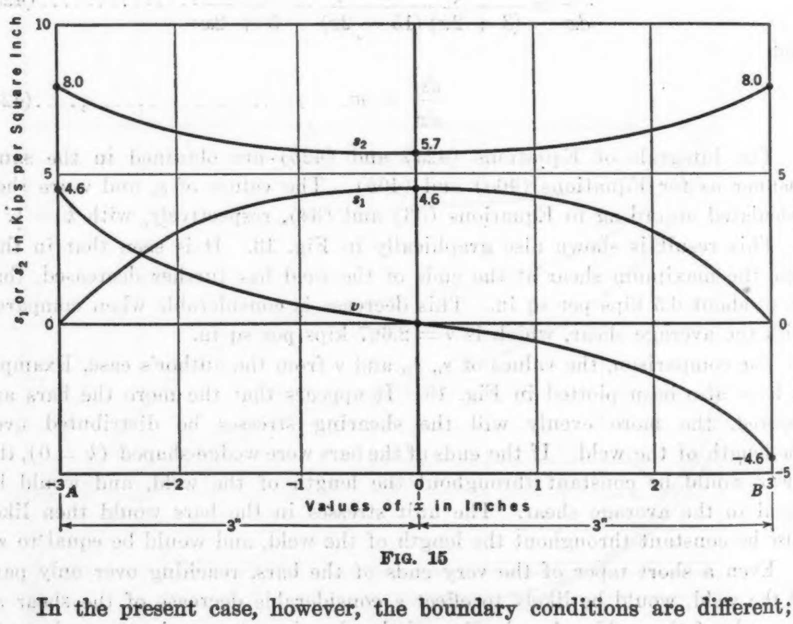

Fig. 15

In the present case, however, the boundary conditions are different; so that $v = 0$ when $x = 0$, and $s_1 = 0$ when $x = \frac{z}{2}$. Therefore, $K_1 = 0$ and $K_2 = \dfrac{P}{\cosh \dfrac{z}{2\,b}\,(a_1 + a_2)}$. Substituting these values in the author's formulas

[18] Tokushima, Japan.

[18a] Received by the Secretary January 27, 1933.

his Equations (23), (24), and (15) may be rewritten, as follows:

$$s_1 = \frac{P}{a_1 + a_2}\left[1 - \frac{\cosh \frac{x}{b}}{\cosh \frac{z}{2\,b}}\right] \qquad (43)$$

$$s_2 = \frac{P}{a_1 + a_2}\left[1 + \frac{a_1 \cosh \frac{x}{b}}{a_2 \cosh \frac{z}{2\,b}}\right] \qquad (44)$$

and,

$$v = -\frac{P\, b\, D \sinh \frac{x}{b}}{a_2\, E \cosh \frac{z}{2\,b}} \qquad (45)$$

If the area, a_1, of the reinforcing plate is 4 sq in., the area, a_2, of the main plate is 8 sq in., and if the other quantities are the same as in the author's Example 1, the resulting shears and tensile stresses will be as shown in Fig. 15.

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852
DISCUSSIONS

WORK OF RIVETS IN RIVETED JOINTS

Discussion

By Messrs. W. P. Roop, H. N. Hill and Marshall Holt, Donald E. Larson, and A. E. R. de Jonge

W. P. Roop,[8] M. Am. Soc. C. E. (by letter).[8a]—It is an opportune moment for re-opening discussion of riveted joints. Comment of two kinds is presented herewith, namely, another solution of the specific problem of this paper, and a more general discussion of experimental methods and data.

The consideration of relative deformations in different parts of a riveted joint is an important departure from practice in conventional design, and one which leads to some rather unfamiliar points of view.

Point Rivets.—The author's conception of a rivet appears to be that of a point or small area in which there is no relative motion of the faying surfaces with respect to each other at loads below limiting friction; when this limit is exceeded relative motion occurs to the extent that resistance due to friction and to elastic deformations in plates and rivet will permit. This elastic resistance is considered to be proportional to the amount of the relative motion, or slip.

From this as a starting point, the author develops formulas for action of rivets in a lap-joint in tension. Couples due to the eccentricity of the tensions in the two plates are ignored, which is more or less equivalent to splitting a double-strapped joint down the center of the plate, so as to obtain only one strap in the picture. Fig. 3 shows the assumed relative deformations.

Diagrams of this kind form a convenient basis for the entire discussion, and their character should be defined a little more clearly. It is preferable to adopt the mid-length of the lap as the point of reference, and in order to obtain complete symmetry this point is taken half-way between the points at mid-length of the two sides of the lap. On Fig. 3 this point would be represented midway between Points Q and W. Distances in the diagram measured vertically from a horizontal line through this point represent horizontal dis-

Note.—The paper by A. Hrennikoff, Esq., was published in November, 1932, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: March, 1933, by Messrs. Henry W. Troelsch, Henry B. Seaman, A. H. Finlay, and F. P. Shearwood.

[8] Lieut. (CC), U.S.N.; New York Shipbuilding Co., Camden, N. J.

[8a] Received by the Secretary January 3, 1933.

placements, and as the diagram is drawn, distances laid off upward represent displacements to the left, and *vice versa.*

A somewhat different graphical representation of this situation is obtained by drawing two parallel lines representing the two sides of the surface of

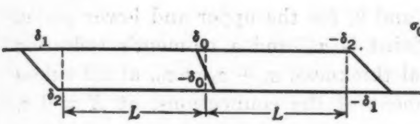

FIG. 9.—DEFLECTIONS

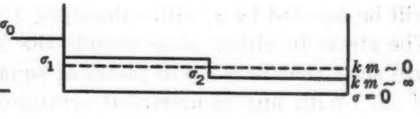

FIG. 10.—STRESSES

contact (faying surface) in the lap. Although actually these two surfaces coincide, the lines representing them are separated, in Fig. 9, by a convenient distance. To each point in one plate, *P*, there corresponds a point, *P′*, in the other plate; in the unloaded condition of the joint these two points coincide, and in Fig. 3 a line joining them would be normal to the base lines, but when the joint is loaded, all these lines are inclined. Conventional design is based on the idea that the inclination, *P P′*, is uniform throughout the joint, so that all rivets are equally loaded. However, this is impossible except in a limiting case, such as that where contact between the plates is at two points only, the load being equally shared at these two points. Otherwise, the relative displacement of one plate with respect to the other is more complex.

In a diagram of the type used by the author the case of two (rows of) point rivets would be represented by two horizontal lines at equal distances above and below the base. The case of three point rivets is that of the author's Example 1.

Graphical interpretation may be given to the results expressed in Equations (9) and (10). When k is small and the rivets are stiff, the lines, $M\,O$ and $S\,U$ (Fig. 3), are nearly straight. The value of a is also affected, and in the limiting case of $k = 0$, becomes equal to $F\dfrac{p}{wtE}$. The middle row of rivets then carries no load, and $M\,O$ and $S\,U$ are horizontal. As k increases, and the connections in the lap become more flexible, $M\,O$ is inclined and N departs from the straight line joining M and O. When $k = 3\dfrac{p}{wtE}$, a has already reached a value slightly negative, namely, $-\dfrac{1}{11}\,F\,\dfrac{p}{wtE}$, and as k continues to increase, a will reach larger negative values, approaching $-\dfrac{kF}{3}$, and the inclination of $M\,O$ will increase indefinitely.

It is for such reasons that the writer prefers another type of diagram instead of that used by Mr. Hrennikoff. In this third type of diagram the horizontal co-ordinate is still length, measured both ways from mid-length of the joint. Let the entire length of the joint be $2L$, and the distance from mid-length to a given section of the joint, ξ; then a convenient co-ordinate to use is

$X = \frac{\xi}{L}$, since it is applicable to a joint of any length, and the joint lies between the limits, $-1 < X < +1$. For a vertical co-ordinate, however, the writer chooses stress in the plate. In the author's notation, this is $\frac{F}{wt}$, and will be denoted by σ, with subscripts, 1 and 2, for the upper and lower plates. The stress in either plate outside the joint is σ_0, and a moment's reflection will show that in a lap of plates of equal thickness, $\sigma_1 + \sigma_2 = \sigma_0$, at all values of X. With any symmetrical arrangement of the connections, at $X = 0$, $\sigma_1 = \sigma_2 = \frac{\sigma_0}{2}$. Between point rivets, σ_1 and σ_2 will have uniform values, so that the curve will be, for σ_1, a series of steps downward from σ_0 at $X = -1$, with a riser at each intervening rivet, landing on $\sigma_1 = 0$ at the last rivet on the right, where $X = +1$.

With two rows of rivets, σ_1 drops immediately at $X = -1$ to $\frac{\sigma_0}{2}$ and retains this value until $X = +1$, when it drops to 0. With three rows of rivets expressions for σ_1 and σ_2 may be obtained by a process similar to that used by the author. The deflection of a point rivet, δ, is proportional to the tensile stress (in the plate) which produces the deflection, thus $\delta = k\sigma$. This tensile stress also produces elongation in the plate according to an equation of the type, $d = \frac{\sigma}{m}$. Thus, k is the inverse stiffness of the rivet in shear, and m is the elastic modulus of the plate. The solution is,

$$\sigma_1 = \sigma_0 \frac{2\,km + 1}{3\,km + 2}$$

When k and m are large, the rivets slender and flexible, and the plates thick, σ_1 is nearly two-thirds σ_0, and each of the three rivets carries about one-third the total load. When k and m are small, the rivets large, and the plate thin, σ_1 is about one-half σ_0, and the middle rivet carries little load. Fig. 10 exhibits this graphically. The stress in the other plate at the same point, X, is σ_2, or, on account of symmetry, it is the stress in the same plate at $X' = -X$.

By similar methods a solution can be found for a joint with any number of rows of point rivets, but the elimination of deflections will require solution of a correspondingly large number of linear equations. This will lead to a uniform value for σ_1 in each interval of plating between rows. These values will drop from σ_0 to 0 by a corresponding number of steps; when $km \to \infty$, the load will be shared about equally by all rivets, and when $km \to 0$, the outer rivets will take practically all of it.

Distributed Connection.—Now the point rivet, on which this entire analysis is based, is a rather crude abstraction. The area of the holes alone in a lap-joint may run to 13% of the area of the lap, and the area under high heads and points may double that figure. Further, the clamping effect of a rivet is surely not limited to the area under the head. In studying this question

the writer has approached it with the assumption that at moderate loads the clamping area is 100% of the lap, and that as slip sets in, it starts at the edges where the stress concentrations occur, and spreads inward. He has further assumed that even in the area of contact the deflections in the parting between the plates are large compared with the variations in stress through the thickness of the plate. Such an assumption is necessary, in order to make any analysis of this character possible, but instead of connection between the plates only at isolated points, assume the elastic resistance to tension to be distributed continuously over the entire area of the lap.

The full analysis for this case has been worked out, but only the results and definition of the quantities will be given herein. The stiffness of the elastic connections must now be expressed in terms of elastic resistance per unit length of the joint (parallel to the load), and also, of course, per unit deflection. The quantity, *k*, will be defined, as before, in an inverse relation, so that a high value of *k* is associated with large deflections under a given load; *m* retains the same meaning as before, being in direct proportion to Young's modulus for the plate, but depending also on dimensions of the joint. The solution is in the form,

$$\sigma_1 = \frac{\sigma_0}{2}\left[1 - \frac{C}{C^2 - 1}(C^X - C^{-X})\right]$$

As before, when $X = 1$, $\sigma_1 = \sigma_0$; when $X = 0$, $\sigma_1 = \frac{\sigma_0}{2}$; but instead of changing by a series of steps, the stress now varies continuously, following a curve the course of which depends on the value of C. This, in turn, depends on the elastic characteristics of the joint according to the equation:

$$C = e^{\sqrt{\frac{1}{km}}}$$

Thus, when km is large, the elastic connection flexible, and the plates thick, C approaches unity, log C approaches zero, and C^X approaches $1 + X \log C$. Then,

$$\sigma_1 = \frac{\sigma_0}{2}\left[1 - \text{Constant } X\right]$$

and since $\sigma_1 = \sigma_0$ when $X = -1$, and $\sigma_1 = 0$ when $X = +1$, the constant must be -1. On the other hand, when km is very small, C is very large, and:

$$\sigma_1 \backsimeq \frac{\sigma_0}{2}\left[1 - C^{X-1} + C^{-X-1}\right]$$

It is easy to see how σ_1 varies with X at various values of km by writing a few numbers. For example, when $km = 0.01$, σ_1 has dropped off to 0.57 at $X = -0.8$; and as $km \backsimeq 0$, the curve would approach a vertical drop to $\sigma_1 = \frac{\sigma_0}{2}$, following along this value until the other end of the joint was approached. In Fig. 11, $\frac{\sigma_1}{\sigma_0}$ is plotted on X for various values of km.

This solution appears to make a somewhat closer approach to the truth than one based on point rivets, but it still departs from the truth in assuming that

the elastic connections between the plates are distributed all over the area of the lap. The connections are doubtless actually somewhat spotty, and actual stress distributions would be affected. Nevertheless, the main feature of the results must hold in actual cases also, such as a concentration at the edges of the lap which always exists, but is more pronounced when plate edges are clamped tightly and the plate itself is relatively extensible. Over-all extensions under load have also been calculated both for point rivets and distributed clamping. Discussion of this matter, however, is reserved for the present.

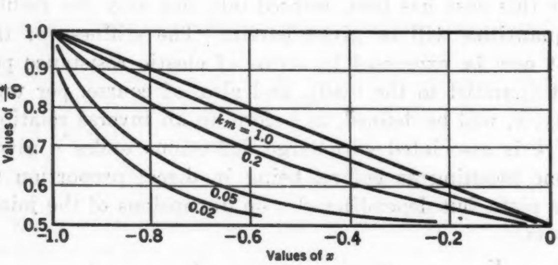

FIG. 11.—DISTRIBUTED CONNECTIONS.

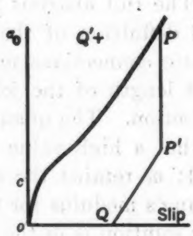

FIG. 12.—SLIP CURVE.

The next step in the analysis of a lap-joint would lie in consideration of the distribution of stress across the thickness of the lapped plates, which obviously cannot be uniform. This would lead still deeper into the theory of elasticity, although the question is also accessible to study by the photo-elastic method. Purely analytical study of riveted joints in this field appears to have about reached its present limit. In fact, the object of such calculations as these is not to obtain conclusions directly applicable in the design of joints, but rather to obtain a nominal analysis sufficiently close to the truth to permit its use in reduction of experimental data and evaluation of empirical constants.

Experimental Studies.—All experiments have from necessity dealt with entire joints and not with such abstractions as point rivets. Perhaps it may become possible to study experimentally the separate elements of a riveted joint, but for the present designers are practically limited to judging the correctness of analysis by the extent to which it leads to correct reconstruction of data observed for whole joints. Such data have been available for a long time as far as ultimate strength is concerned, but, unfortunately, other data which are of more significance than the ultimate strength, have been obtained only to a limited degree. By this are meant in particular data on stresses at working loads, and on elongations. That these are more significant than ultimate strength of isolated joints will be apparent from the consideration that (at least in ships) a joint seldom works in isolation, but is almost always in combination with other elements of construction in such a way that relative deformations are of primary significance. Further, after elastic limits have been passed, even an isolated joint works in an entirely different way from that by which working loads are resisted. No doubt the basic assumption of

conventional design, that all rivets share the load equally, has some justification as far as the ultimate strength of a lap-joint goes. For such a joint as that shown in the author's Fig. 2, however, such an assumption is not even approximately correct, and even in the simple lap-joint, under working loads, inequalities in rivet loading, such as the author points out, have important consequences.

Experiments in which elongation and stress distribution were observed, were contemplated as long ago as 1869.[9] Known to the writer in detail are only two series of tests in which serious efforts to observe elongations were made, namely, those of Montgomerie,[10] and those made at the Bureau of Standards by Commander E. L. Gayhart, for the Bureau of Construction and Repair, U. S. Navy Department,[11] which will be referred to as the "1924 tests." No doubt similar studies have been made, and are known to others.

In all discussion of the elongation of riveted joints prominence is given to the word, "slip." It is important that its meaning should be agreed upon. In early tests a fine line was scribed across the parting at the plate edges and observed with a glass. When the continuity of this line was observed to be broken, slip was said to have begun. Close observations of slip on this basis, taken on a series of lines spread over the length of the joint, with the help of a micrometer microscope, might produce results of value, and as far as the writer knows this has never been undertaken.

Since over-all elongation of the joint is, for many purposes, the significant quantity, slip has sometimes been taken to be the remaining elongation on release (but not reversal) of the tensile load. This is based on the idea that this phenomenon is due to having exceeded a critical frictional resistance somewhere, which permitted sliding of the plates over each other; release of load would not be sufficient to remove such a deformation which would thus appear as a permanent set. For observation of slip on this basis, however, release is necessary, leading to complicated re-adjustments. For this reason a third view of slip was taken in the 1924 tests.

Instead of simple over-all elongation, deformation was measured as between a number of pairs of points; curves were plotted on load, and a break in the curve, such as that marking proportional limit in a tensile test specimen, was taken to mark the beginning of slip. The amount of slip at a given load between a given pair of points was obtained by subtracting from the extension as observed by a dial gauge a calculated correction for the elastic part of this extension. This calculation was on a rather nominal basis, no great emphasis being placed on its absolute value in view of the numerous other uncertainties involved.

In order to eliminate as far as possible arguments based, not on real differences of opinion, but on differences of definition, the writer proposes that slip be regarded as the kind of a quantity actually observed in the early tests

[9] "Shipbuilding in Iron and Steel," by E. S. Reed; also, "On the Present State of Knowledge as to the Strength and Resistance of Materials," by Jules Gaudard, *Minutes of Proceedings*, Inst. C. E., Vol. XXVIII (1868–69), pp. 536–571, and Vol. XXIX (1869–70), pp. 25–97.

[10] *Transactions*, Inst. of Engrs. and Shipbuilders in Scotland, 1920; also, *Transactions*, Inst. of Naval Archts., 1923.

[11] *Transactions*, Soc. of Naval Archts. and Marine Engrs. (1926), p. 55.

on the plate edges, namely, the distance separating two points on the two sides of the faying surface which were originally coincident. On this basis slip will depend on the entire history of the joint, so that "original condition" means whatever starting point was adopted for the observations. On this basis, slip throughout the area of the joint is not directly amenable to observation. Such data on slip as are inferred from observations should be qualified; thus "edge" slip explains itself. Inferred slip based on the subtraction of a calculated elastic correction might be called "nominal" slip. As defined, slip will vary from point to point throughout the joint; values obtained from data on the extension of the whole joint might be called 'over-all" slip. Perhaps the most important distinction of all is that between residual slip, which is that remaining on release of load, and elastic slip which, generally speaking, would be in direct proportion to the load. More specifically, elastic slip would be defined as the difference between the total slip and the residual slip, relative motion of the plates being under control of the elastic resistance of the rivets.

Armed with this specific view of the nature of slip, the reader may return to view Part III of the author's paper, devoted to "Numerical Values of the Forces of the Rivets."

Accepting the author's demonstration of overstressing in outer rivets, it still seems fair to say that the picture bears little resemblance to the facts until phenomena of friction and slip are considered. Commander Gayhart has shown[12] that over-all slip begins at loads that may well lie within working ranges. The curves as drawn in his paper require attention to the manner in which they are constructed. A type curve is shown in Fig. 12, in which it should be noted that the point plotted at Q' is obtained only at zero external load; it is actually located at Q, but is placed at Q' for convenience. During release the course of the curve is from P to Q. The vertical descent, $P\,P'$, represents the release of the frictional resistance. However, before the elongation can be reduced, as in passing from P' to Q, a reversed friction must be overcome. The friction, $P\,P'$, therefore, might be expected to equal twice the critical friction, $O\,C$. In the case shown in Commander Gayhart's paper it is much greater than this.

Beyond the point at which over-all slip begins, not only elastic slip, but residual slip, increases approximately in direct proportion to total load. These facts lead to questions as to the details of slip action, which must be settled somehow before a theory of riveted joints has even a chance of being correct.

If it could be assumed that friction played no part at loads above that of initial slip, or if the residual slip could be assumed equal to zero, the theory developed by the author would be applicable to the idealized cases considered. Such assumptions, however, are obviously wide of the mark. It is not possible to separate elastic slip from residual slip and then treat the elastic slip as if residual slip played no part in determining elastic reactions. The true analysis of these curves must be based on a separation, not of the elements of slip under a given total load, but of the elements of load under a given total slip.

[12] See *Transactions*, Soc. of Naval Archts. and Marine Engrs. (1926), Pl. 22, Sheet 3.

Summary and Conclusion.—The slip curve which has been discussed is not the only type that was obtained in the 1924 tests, but this is not the place for extended discussion of those tests. The principal object of this comment has been to show that rivet action, even to the first degree of approximation, may be considerably more complicated than is contemplated by the author. If sound progress beyond what he aptly calls "conventional design" is to be made, it is essential that more experimental information be made available, and especially that it should receive a searching analysis, with subsequent tests especially devised to provide answers to the questions developed by the analysis. When the results of such study become accessible to those who are directly responsible for detail design the Engineering Profession may perhaps hope to see no more such absurdities as that mentioned by the author—a tensile joint with twenty rows of rivets.

H. N. Hill[13] and Marshall Holt,[14] Juniors, Am. Soc. C. E. (by letter).[14a]—The unequal distribution of the load on the rivets in a joint is a matter deserving of greater recognition than is ordinarily accorded it by the Engineering Profession. This distribution has been the subject of an investigation conducted by the writers under the supervision of R. L. Templin, M. Am. Soc. C. E. The investigation concerned the distribution of load in butt-joints comprising splices between plates of steel and a high-strength aluminum alloy, using both steel and aluminum alloy rivets.

The mechanical properties of the two metals used in the investigation are given in Table 3. It may be noted that there is little difference between the strength of the two materials, but that the modulus of elasticity for the aluminum is about one-third that for steel. The problem involved the distribution of the load on the various rivets, not only as affected by the physical dimensions of the joint, but also as affected by the different values of the modulus of elasticity for the two materials.

TABLE 3.—Mechanical Properties of Steel and High-Strength Aluminum Alloy

Properties	Steel	Aluminum alloy
Ultimate tensile strength, in pounds per square inch	59 000	56 000
Yield strength in tension, in pounds per square inch	28 000	33 000
Percentage elongation in 2 in.	37.5	23.0
Modulus of elasticity, in pounds per square inch	29 000 000	10 000 000
Shear strength, in pounds per square inch	45 000	35 000
Shearing modulus, in pounds per square inch	12 000 000	3 800 000

The problem was attacked in two ways: First, an "elastic theory" was developed, along lines similar to those described by Mr. Hrennikoff, taking into account the elastic deformation of the various parts of the joint and assuming the load transmitted by each rivet to be proportional to the relative

[13] Research Structural Engr., Aluminum Co. of America, New Kensington, Pa.

[14] Research Structural Engr., Aluminum Co. of America, New Kensington, Pa.

[14a] Received by the Secretary February 1, 1933.

displacement of the plates; and, second, a number of double-strap butt-joints, embodying a single longitudinal row of rivets, were fabricated and tested to determine experimentally the manner in which the load was distributed among the various rivets.

In establishing a relationship between the deformation and the load on a rivet, the writers have not felt that the limits of accuracy, dictated by the uncertainties existing in any joint, warrant an elaborate mathematical treatment. An experimental determination of the relationship between deformation and load would be of considerably more value than any elaborate mathematical analysis, particularly since the theoretical treatment would require the substantiation of experimental data to make it convincing.

As a basis for the calculations in the mathematical treatment, a relationship between the load and deformation on a rivet was obtained from a preliminary test of a joint, similar to those to be investigated, but having only one rivet on either side of the splice. This specimen was fabricated entirely of aluminum alloy, and consisted of two main plates, 1 in. thick by 5¾ in. wide, two cover-plates of the same width and ½ in. thick, and two ¾-in. rivets. The relative movement of the plates was measured for different loads on the rivets. Although for low loads and relatively high loads, the load-deformation curve was not a straight line, for loads corresponding to the range to be encountered under normal working conditions, differences in the slopes of lines drawn through zero from points on the curve corresponding to the different loads are not great enough to produce an appreciable effect on the results of the calculations.

The results of this test established a relationship between the load and deformation of an aluminum alloy rivet in plates of the same material, but did not cover the other conditions in the joints to be tested. Since the deformation is partly produced by yielding of the plates, a definite load on an aluminum rivet in aluminum plates will produce a greater relative movement than would occur if the plates were of steel. Since the modulus of elasticity of aluminum, is approximately one-third that of steel, it is reasonable to assume that, in a steel specimen of the same dimensions as the one tested, the ratio of deformations would be 1 to 3.

In Table 4 the ratios of load to deformation for conditions other than those covered by this test have been determined by estimating the effects of differences in the conditions. Familiarity with the calculations for determining the distribution of the load among the various rivets reveals the fact that the answer is not extremely sensitive to variations in the ratio of load to deformation of a rivet. Any reasonably correct ratio of load to deformation will give results that are within the range of accuracy to be expected from variations introduced in the fabrication of a particular joint.

It is interesting to note the agreement between the coefficient determined experimentally by the writers and that calculated from the author's equations for the same case. The value calculated according to the author's equations for the second set of conditions (Table 4) was 0.000154 in. per 1 000-lb load, whereas a value of 0.00012 was obtained by the writers as one-third of

the relative movement determined experimentally for the first set of conditions of Table 4, one-third being the ratio of the modulus of elasticity of aluminum to that of steel.

TABLE 4.—RELATIONS BETWEEN DEFORMATION AND LOAD ON RIVETS

Thickness of plates, in inches	Main plate	Cover-plate	Rivets	Coefficient $\left(\frac{\text{Inches}}{1\ 000\ \text{lb}}\right)$
1 and $\frac{1}{2}$......	Aluminum	Aluminum	Aluminum	0.00035
1 and $\frac{1}{2}$......	Steel	Steel	Steel	0.00012
1 and $\frac{1}{2}$......	Steel	Aluminum	Steel	0.00014
1 and $\frac{1}{2}$......	Steel	Aluminum	Aluminum	0.00030
1 and $\frac{1}{2}$......	Aluminum	Aluminum	Steel	0.00017
$\frac{3}{4}$ and $\frac{3}{8}$......	Aluminum	Aluminum	Steel	0.00013
$\frac{3}{4}$ and $\frac{3}{8}$......	Steel	Aluminum	Steel	0.00010

Three large riveted specimens were tested, representing splices between a steel plate and one of aluminum. The specimens were $5\frac{3}{4}$ in. wide, using aluminum cover-plates and embodying five $\frac{3}{4}$-in. rivets on either side of the splice. The rivets were spaced on 4-in. centers. Details of the three specimens are given in Table 5.

The distribution of the load on the various rivets was determined by measuring the stresses existing in the cover-plates at sections between the rivets, the difference between the load on two adjacent sections being a measure of the load transmitted by the intervening rivet. Strains were measured with a 2-in. Berry strain-gauge on twenty-four gauge lines at each section between two adjacent rivets. The relatively low modulus of elasticity of aluminum permits the accurate determination of comparatively low stresses, the strain in aluminum being about three times as great as that produced by the same stress in steel. The joints were investigated for loads ranging to somewhat above the nominal design load of the specimen, Specimen No. 2, Table 5, being loaded in both tension and compression.

A summary of the results of the tests is shown in Table 5, together with the calculated values of the percentage of the load transmitted by each rivet for a load corresponding to a stress slightly less than the nominal working stress of the joints. The rivets are numbered from 1 to 5, No. 1 being the outer rivet on either end of a joint. Where the main plate and the cover-plates are of the same material and the latter is one-half the thickness of the former, the loads transmitted by the end rivets (Nos. 1 and 5) have been averaged, as have the loads for Rivets Nos. 2 and 4. In the case of the aluminum cover-plates on the steel main plate, however, each rivet transmits a different part of the load (because of the difference in the values of the modulus of elasticity), and the values for each rivet have been listed individually.

Considering the uncertainties that are introduced in the fabrication of such a joint, the results shown in Table 5 indicate a fair agreement between the calculated and measured values for the percentage of load transmitted by the various rivets. For the case of a steel main plate, aluminum cover-plates, and steel rivets, the calculated values indicate a more serious condition of over-load on the outer rivet than was measured experimentally.

TABLE 5.—PERCENTAGE OF TOTAL LOAD CARRIED BY EACH RIVET IN A LARGE BUTT-JOINT
(At Nominal Working Load)

Specimen No.	Thickness of main plates, in inches	Thickness of cover-plates, in inches	Kind of rivets	Determination	Aluminum Main Plate Rivets Nos. 1 and 5	Aluminum Main Plate Rivets Nos. 2 and 4	Aluminum Main Plate Rivets Nos. 3	Steel Main Plate Rivets Nos. 1	Steel Main Plate Rivets Nos. 2	Steel Main Plate Rivets Nos. 3	Steel Main Plate Rivets Nos. 4	Steel Main Plate Rivets Nos. 5
1......	¾	⅜	Steel	Experimental..	34	13	6	17	8.5	12.5	27.5	34.5
				Calculated.....	31.5	14	9	17	9	10	19.5	44.5
2......	1	½	Steel	Experimental	29.5	14	13	23	14.5	16	22	24
				Experimental	*25.5	19	11	16	17	16	19	32
				Experimental	†25.5	18	13	18.5	9.5	13.5	23.5	35
				Calculated.....	28	16	12	17	12	13	20	38
3......	1	½	Aluminum	Experimental..	24.5	19	13	19.5	10	19	21.5	30
				Calculated.....	24.5	18	15	17	15	16	21	31

* Specimen straightened before test. † Specimen loaded in compression.

Specimen No. 3 failed by shearing the rivets at a load of 151 500 lb, which corresponds to the calculated strength of the joint, calculated as the total shear strength of all the rivets. This fact suggests that at the breaking load each rivet was carrying an equal share of the load.

The frictional resistance of a riveted joint, as a factor in its behavior, is frequently given considerably more importance than it merits. According to the theory of static frictional resistance, load on the joint is resisted by frictional forces between the plates, and is accompanied by no relative movement of the adjacent faces of the plates. Mr. Hrennikoff discusses the action of the static friction in a joint having more than two rivets in a longitudinal row. His conclusion is correct that "a method of design based on allowing a uniform value for each rivet, working in friction [as is done in designing for shear], is wrong." However, the statement that only the outer rivets are effective in static friction, the intermediate rivets performing no work, is not entirely true. To maintain static friction, it is essential that there be no relative movement of the adjacent faces of the plates. This does not mean that the length of the plate, $A_1 B_1$, must be the same as that of the plate, $A_2 B_2$ (Fig. 1(*c*)). It is entirely possible for the two adjacent faces of the plates between *A* and *B* to remain the same length and yet for the shear detrusions be so distributed throughout the thickness of the plates as to produce average changes of length that are different for the two plates. In the case of the lap-joint shown in Fig. 1(*c*), this phenomenon would be accompanied by bending of the plates, which is just what occurs when such a joint is subjected to a tensile load. It is possible, therefore, for the inner rivets to function in static friction before slip has occurred at the outer rivets. It is true that the percentage of the load that would be resisted by static friction of the inner rivets would be relatively small and, for practical purposes, the assumption that the outer rivets carry all the load (in static friction) is probably not far wrong.

Tests made on Specimen No. 2, Table 5 (which was fabricated with hot-driven steel rivets) revealed slipping in the joint even at very low loads. The results of these tests are shown in Fig. 13. Measurements were made

of the change in length of the cover-plates between the first rivets on either side of the splice and over the gap between the two main plates. If the load were supported entirely by friction in the joint, the change in distance measured over the gap should be about the same as that measured in the

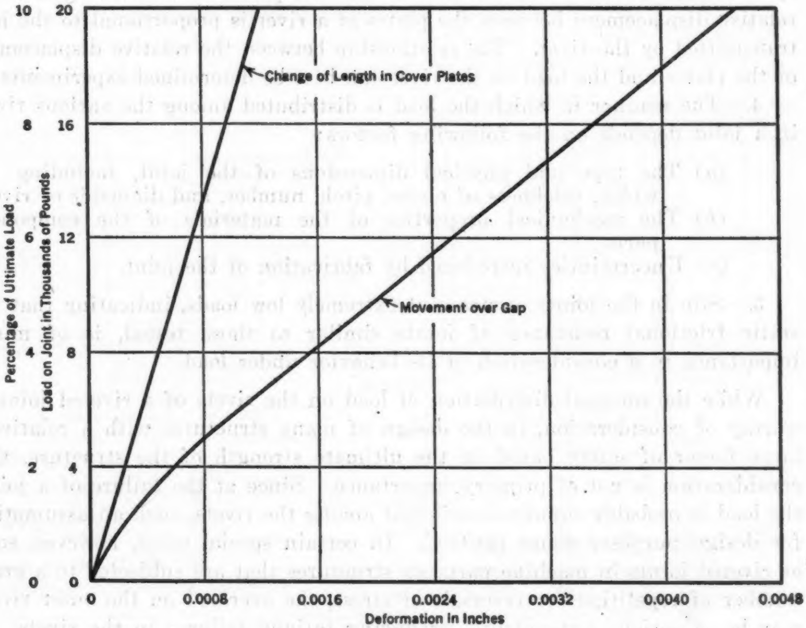

FIG. 13

cover-plates. Fig. 13 shows that the change in distance measured over the gap was relatively greater for extremely low loads, indicating a slip between the plates with the consequent loading of the rivets. Experience has shown that, in general, the friction between aluminum and aluminum, and between aluminum and steel (which are the conditions in Specimen No. 2, Table 5) is greater than the friction developed between steel and steel under the same circumstances. The frictional resistance in Specimen No. 2, therefore, was probably greater than would have been encountered in a similar all-steel joint.

From the results of the investigation, the following conclusions were drawn:

1.—At the working load, the load on a riveted joint is not equally distributed among the rivets in a longitudinal row, as is customarily assumed in design. In a double-strap butt-joint having cover-plates of one-half the thickness of the main plates, the end rivets on either half of the joint carry the greatest load, when the plates are of the same material. In the case of a steel main plate and aluminum cover-plates the first and second rivets from the splice are most highly stressed.

2.—At the breaking load, each rivet carries an equal share of the load, provided the rivets are the same size and of the same material.

3.—The problem of determining the distribution of the load among the various rivets of a joint such as those tested, can be solved mathematically, by considering the elastic deformations of the plates and assuming that the relative displacement between the plates at a rivet is proportional to the load transmitted by the rivet. The relationship between the relative displacement of the plates and the load on the rivet can best be determined experimentally.

4.—The manner in which the load is distributed among the various rivets in a joint depends on the following factors:

(a) The type and physical dimensions of the joint, including the width, thickness of plates, pitch, number, and diameter of rivets.
(b) The mechanical properties of the materials of the component parts.
(c) Uncertainties introduced by fabrication of the joint.

5.—Slip in the joints occurred at extremely low loads, indicating that the static frictional resistance of joints similar to those tested, is of minor importance in a consideration of its behavior under load.

While the unequal distribution of load on the rivets of a riveted joint is worthy of consideration, in the design of many structures with a relatively large factor of safety based on the ultimate strength of the structure, this consideration is not of primary importance. Since at the failure of a joint, the load is probably equally distributed among the rivets, such an assumption for design purposes seems rational. In certain special cases, however, such as riveted joints in machine parts, or structures that are subjected to a great number of repetitions or reversals of stress, the overload on the outer rivets may be of serious consequence, producing fatigue failures in the rivets. A rational design under such conditions should consider the unequal distribution of the load among the various rivets.

DONALD E. LARSON,[15] JUN. AM. SOC. C. E. (by letter).[15a]—An interesting analysis of the deformations and stresses in riveted joints is presented in this paper. Such an analysis reveals the fact that all the rivets in certain types of joints cannot possibly be stressed equally at working loads. In spite of this fact, nearly all riveted joints are designed in accordance with the assumption that the rivets are stressed alike, and this leads to a simple and convenient method of design. No design procedure, however, can be justified on the basis of simplicity if it does not provide adequately for the stresses to which the member is to be subjected. The real justification for proportioning joints on the basis of equally stressed rivets lies in the probability that, before failure occurs, the overstressed rivets will yield and thus will effect a more uniform distribution of load. It follows that the theoretical ultimate strength of a joint designed on the basis of equally stressed rivets can only be attained when the rivets are made from a ductile material. If they were made of a material that obeyed Hooke's law up to the point of failure, as

[15] Asst. Research Engr., Chicago Bridge & Iron Works, Chicago, Ill.

[15a] Received by the Secretary February 18, 1933.

some materials do, it would be necessary to design all joints by some method similar to that outlined by the author. This design procedure would result in the selection of types of joints differing radically from those now in common use.

While it seems probable that the assumed condition of equally stressed rivets is actually attained before failure occurs, it is interesting nevertheless to speculate on the distribution of stress at working loads, and it is to be regretted that the author did not extend his study to include the more com-

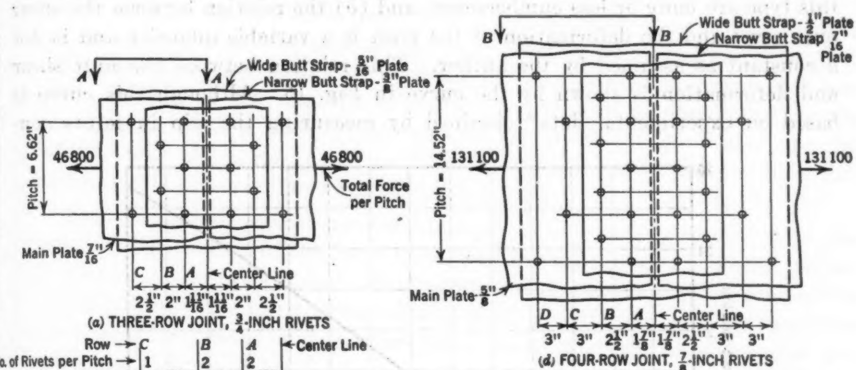

FIG. 14.—STANDARD TYPES OF BUTT-JOINTS

mon types of joints used in the construction of large oil tanks, elevated water tanks, and pressure vessels. Standard butt-joints having three, four, and five rows of rivets are shown in Fig. 14. The rivet pitch shown for each of these joints is that which gives the maximum efficiency for the following assumed conditions: (1) All the rivets are equally stressed; (2) the effective diameter of each rivet is equal to the nominal diameter plus $\frac{1}{16}$ in.; (3) the

value of the plate in tension = 1.0; (4) the value of the rivet material in shear = 0.75; and (5) the value of the rivet material in bearing = 1.5. Theoretically, the strength of the net section of the main plate is equal to the shearing strength of the rivets for each joint.

In making an analysis of these joints to determine the stresses on the various rivets at working loads, the algebraic method used by the author was abandoned in favor of a method of successive approximation for the following reasons: (*a*) The algebraic expressions for the deformations in joints of this type are more or less cumbersome; and (*b*) the relation between the shear on a rivet and the deformation of the rivet is a variable quantity and is not a constant as assumed by the author. This relation between the unit shear and deformation is shown by the curve in Fig. 15. Although this curve is based on experimental data[16] obtained by measuring the slip in joints con-

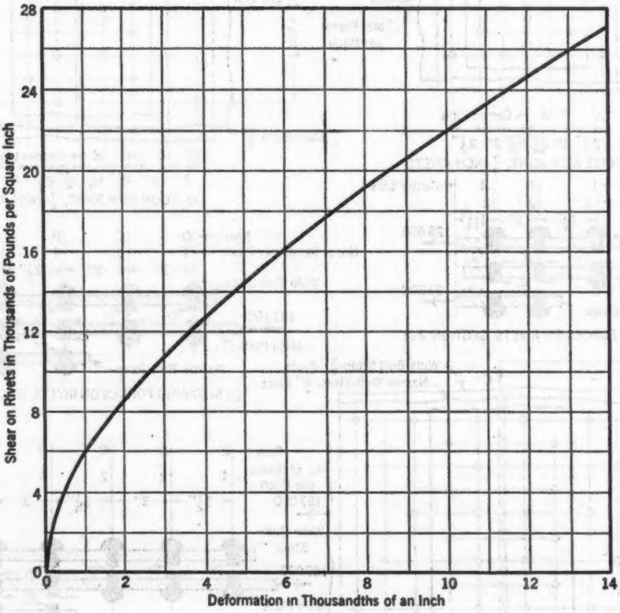

FIG. 15

taining ⅞-in. rivets, it has been used for determining the deformations of the larger rivets in the joints shown in Fig. 14, because of the lack of actual experimental data for these sizes. The rivet stresses are preferably determined as follows:

Step 1.—Apply a force to the main plate of the joint and, assuming that this force is equally distributed among the rivets, compute the tensile stresses in the main plate and in each of the butt-straps between successive rows of rivets.

[16] See Fig. 39 in "Tests of Joints in Wide Plates," *Bulletin No. 239,* Eng. Experiment Station, Univ. of Illinois, Urbana, Ill.

Step 2.—Using stresses from Step 1 compute the elongation of the main plate and of each of the butt-straps between successive rows of rivets; then, from these elongations, compute the amount that each rivet must deform to preserve continuity.

Step 3.—Using rivet deformations from Step 2, determine from Fig. 15 the unit shear that must act on each rivet to produce the given deformation; and from these unit shears compute the total shear on each rivet.

Step 4.—Using the new set of rivet shears from Step 3, recompute the tensile stresses in the main plate and in each of the butt-straps between successive rows of rivets; then using this new set of stresses repeat Steps 2, 3, and 4 until the same results are obtained twice successively. When this condition is reached the total force acting on the joint is distributed among the rivets in such a manner that static equilibrium is maintained and the deformations of the rivets, the main plate, and the butt-straps are such that continuity is preserved in all parts of the joint.

The procedure outlined can be hastened considerably by guessing at the probable distribution of the force applied to the main plate in Step 1 instead of assuming that it is equally distributed among the rivets. Of course, the results obtained are exactly the same as those obtained by writing equations for the deformations and solving them simultaneously to determine the unknown.

To each of the joints shown in Fig. 14, a force that will produce a unit shearing stress of 10 000 lb per sq in. on each of the rivets, if equally distributed among them as assumed in the design, has been applied. The actual distribution has then been computed by the four steps outlined, and the results are shown in Table 6. Each part of the table, 6(*a*), 6(*b*), and 6(*c*), shows the total shear acting on one rivet in each row, the unit shear on that rivet, and the percentage by which this unit shear differs from the 10 000-lb unit shear for which the rivet was designed. This analysis shows that for each of the three joints the rivets in the outer row are the ones most highly stressed. The stress on the outer rivet of the three-row joint is 14% greater than that for which it was designed, whereas the outer rivet of the four-row joint is 34% overstressed, and that of the five-row joint is 42% overstressed. The percentage of over-stress on the rivets in the outer row increases as the number of rows of rivets is increased.

Although it is evident from the foregoing analysis that the rivets in the outer rows of standard butt-joints are greatly overstressed at working loads, it can be stated definitely that this condition does not seriously reduce the ultimate strength of the joint, providing the rivets are made of a ductile material. This statement is based on experimental data obtained from tests[17] of joints in plates having a width of 6 ft. Two three-row joints having the exact dimensions shown in Fig. 14(*a*) and Fig. 14(*b*) developed an average ultimate strength equal to 98% of the theoretical strength computed from the control specimens, on the assumption that all the rivets were equally stressed. Both these test joints failed by tearing of the main plate and not by shearing of the rivets. Similarly, four, four-row joints having the dimensions shown

in Fig. 14(*c*) and Fig. 14 (*d*) developed an average ultimate strength equal to 92.5% of the theoretical strength determined in the same manner. Two of these joints failed by shearing of the rivets and the other two failed by tearing of the main plate. These tests indicate that the actual ultimate strength attained by the joints are fairly consistent with the theoretical

TABLE 6.—ACTUAL DISTRIBUTION OF A FORCE PRODUCING A UNIT (DESIGN) SHEARING FORCE OF 10 000 POUNDS PER SQUARE INCH ON EACH RIVET

Item No.	Description	SHEARING FORCES (SEE FIG. 14)						
		A_1	A_2	B_1	B_2	C	D	E
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
	((*a*) THREE-ROW JOINT, $\frac{3}{4}$-INCH RIVETS (SEE FIG. 14((*a*) AND FIG. 14(*b*))							
	Shear on One Rivet:							
1	Total, in pounds........	4 925	5 200	4 925	5 400	5 900	......	....
2	Unit, in pounds per square inch.................	9 480	10 000	9 480	10 400	11 400	......	....
3	Percentage difference from unit design shear on 10 000 lb per sq in	—5.2	0	—5.2	+4.0	+14.0	......	....
	(*b*) FOUR-ROW JOINT, $\frac{7}{8}$-INCH RIVETS (SEE FIG. 14(*c*) AND FIG. 14(*d*))							
	Shear on One Rivet:							
4	Total, in pounds........	6 700	6 825	6 500	6 600	7 700	9 200	....
5	Unit, in pounds per square inch.................	9 700	9 900	9 400	9 550	11 200	13 400	....
6	Percentage difference from unit design shear of 10 000 lb per sq. in...........	—3.0	—1.0	—6.0	—5.5	+12.0	+34.0	....
	(*c*) FIVE-ROW JOINT, 1-INCH RIVETS (SEE FIG. 14(*e*) AND FIG. 14(*f*))							
	Shear on One Rivet:							
7	Total, in pounds........	8 350	8 675	7 875	8 850	9 100	10 600	12 600
8	Unit, in pounds per square inch.................	9 400	9 730	8 820	9 950	10 200	11 900	14 200
9	Percentage of difference from unit design shear of 10 000 lb per sq in.	—6.0	—2.7	—11.8	—0.5	+2.0	+19.0	+42.0

strengths computed by the usual method in which the rivets are assumed to be equally stressed. Although it would be better design practice to proportion riveted joints in such a manner that all rivets would be equally stressed at working loads as well as at the point of failure, the tests cited herein seem to indicate that the usual method of design provides joints having the required ultimate strength, and, for this reason, it does not seem likely that the present design method will be replaced by more exact methods in the near future.

A. E. R. DE JONGE,[18] M. AM. SOC. C. E. (by letter).[18a]—The problem dealt with by the author is not a new one. When such an old problem is discussed again one expects that either some new results are found, or some new and simpler method of dealing with it is established. The writer proposes to discuss how Mr. Hrennikoff's paper measures up to these two standards.

[17] "Tests of Joints in Wide Steel Plates," *Bulletin No. 239*, Eng. Experiment Station, Univ. of Illinois, Urbana, Ill.

[18] With Babcock & Wilcox Co., New York, N. Y.

[18a] Received by the Secretary March 14, 1933.

At an early date it was realized that the distribution of the load on a riveted joint over the various rivets is not uniform. This seems to have been clearly expressed for the first time as far back as 1881 by the late W. C. Unwin,[19] Hon. M. Am. Soc. C. E., in England, and again, in 1884, by Th. Landsberg[20] in Germany.

As far as the writer is aware, the first to state definitely that the outer rivets of lap-joints and the outermost and innermost rivets of double-butt strap joints transmit a greater part of the load than the intermediate rivets, was J. T. Milton,[21] in England (1885).

During some tests made by G. Lanza,[22] in 1887, at the Watertown, N. Y., Arsenal, of the United States Ordnance Department, an attempt was made to find the distribution of load on the various rivets from the uneven elongation of the rivet holes.

In 1892, C. Bach[23] in Germany, showed from tests that the resistance of the rivets in the various rows was not the same due to the unequal distribution of the force transmitted by the joint. In England, C. E. Stromeyer,[24] discussed, in 1893, the problem of the partition of load on the various rows of rivets, citing Milton's experiments.[21]

The same subject was discussed in the United States in 1902 by W. H. Boughton,[25] Assoc. M. Am. Soc. C. E., who investigated the question as to whether or not, in the light of the knowledge of an unequal distribution of the load over the various rivets, the usual rivet connections required changing. His answer was that no changes were required.

A discussion was started in the United States in 1904 by an anonymous letter in *Engineering News*,[26] on the same subject, in the course of which the Editor drew attention to the lack of systematic experiments on the subject.

The first, to the writer's knowledge, to undertake a rigid mathematical analysis of the subject was Ivan Arnovljevič,[27] who in Austria, in 1908 and 1909, derived general formulas for the load transmitted by each rivet of a double-butt strap joint. While it is true that Arnovljevič only treated butt-joints, his method is quite general and is equally applicable to lap-joints. He showed that the two outermost and the innermost rivets of each half of a butt-joint transmit most of the load, while the intermediate rivets transmit only a very small part of it. He further showed that on account of this distribution more than five rivets in line are useless, since the additional rivets transmit insignificant fractions of the load only. In addition, he dealt

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[19] *Proceedings*, Inst. of Mech. Engrs. (London), pp. 301-368.

[20] *Centralblatt der Bauverwaltung*, Vol. 4, May 17, 1884, pp. 201-203.

[21] *Transactions*, Inst. of Naval Archts. (London), Vol. 26 (March 27, 1885), pp. 204-210.

[22] Rept. of the Tests of Metals and Other Materials for Industrial Purposes Made at the Watertown Arsenal, 1887, pp. 882-923.

[23] *Zeitschrift* des Vereins Deutscher Ingenieure, Vol. 36 (October 1, 1892), pp. 1141-1148; and Vol. 36 (November 5, 1892), pp. 1305-1314.

[24] "Marine Boiler Management and Construction," by C. E. Stromeyer, Longmans Green & Co., (London), 1893.

[25] *Proceedings*, Ohio Soc. of Surv. and Civ. Engrs., 1902, pp. 17-26.

[26] *Engineering News*, June 9, 1904, pp. 542 *et seq.*

[27] *Oesterreichische Wochenschrift für den öffentlichen Baudienst*, Vol. 14 (August 22, 1908), pp. 607-615; also, *Zeitschrift für Architektur and Ingenieurwesen*, Vol. 50 (Old Series), Vol. 14 (New Series) (1909), pp. 89-106.

with bracket-plate and chord girder plate connections. In his conclusions, Arnovļevič definitely states the laws as revealed by his investigation and gives finally the following recommendations:

1.—The rivet pitch in the direction of the bar should be as small as possible.

2.—Fewer rivets of a large diameter are more advantageous than more rivets of a small diameter.

3.—More than five rivets in a line parallel to the axis of the bar are useless.

4.—Excessively strong butt straps impair the strength of the joint.

5.—The connection of truss members by means of gusset-plates is more advantageous than their connection to the web-plates of chord girders.

His solution is very elegant and quite general. It allows of the checking of the formulas by the determination of the "rivet factor" from experimental results, and thus his investigation opened the way to a rational treatment of riveted joints in general, which, however, was unfortunately never attempted.

Tests made in 1911 by Max Rudeloff,[28] in Germany, and reported on again later by F. Kögler[29] showed the unequal distribution of the load by the unequal elongation of the holes.

William H. Burr, M. Am. Soc. C. E., in the United States, likewise discussed[30] this problem. (See the Sixth (1904) and Seventh (1915) Editions of Professor Burr's book.)

In 1916, R. N. Blackburn[31] published in the United States a study on unsymmetrical riveted boiler joints and showed that these joints (lap-joints and butt-joints with a wider inner than outer butt strap) are less efficient than symmetrical double-butt strap joints due to the unequal distribution of the forces and stresses.

In 1916, Professor Cyril Batho,[32] of McGill University, Montreal, Que., Canada, made an exceedingly careful investigation of the partition of load in riveted joints and derived, by means of the principle of least work, theoretical formulas for the load transmitted by the various rivets. He checked his theoretical results by experiments in which the strain at various points of the joints was measured by means of very sensitive strain-gauges. The result of these experiments proved to be in good agreement with his theory. He thus proved that a riveted joint may be considered as a statically indeterminate structure and that the loads carried by each rivet may be obtained by the principle of least work in terms of a quantity, K, which depends on the manner in which the work is stored in the joint. He determined this coefficient, K, both theoretically and experimentally, finding fair agreement. He also considered the influence of an unequal distribution of the stresses in the plates, both transversely and longitudinally, the unequal distribution of the load between the two cover-plates, and the influence of the

[28] *Verhandlungen des Vereins zur Beförderung des Gewerbefleisses*, Supplement to Vol. 90 (1911), pp. 1-82; see, also, the photograph given.

[29] Berichte des Auschusses für Versuche im Eisenbau, Edition B, No. 1, 1915.

[30] "The Elasticity and Resistance of the Materials of Engineering," by William H. Burr, John Wiley & Sons, N. Y., 1915.

[31] *Power*, Vol. 44 (August 8, 1916), pp. 203-206.

[32] *Journal*, Franklin Inst., Vol. 182 (November, 1916), pp. 553-604.

difference of the modulus of elasticity of the cover-plates to that of the main plate, as well as other types of connections.

Professor Batho's results, although obtained in a totally different way, agree well with those of Arnovļevič.[27] Professor Batho also showed how to calculate the distribution of load in lap-joints and thus goes a step further than Arnovļevič. He has covered, therefore, the entire field of riveted joints as used in structural engineering.

In Austria, Paul Fillunger[33] again treated this subject in 1919, but considered the rivets as mere points being completely rigid while the plates only were subject to deformation. His results are somewhat similar to those of Arnovļevič[27] and Batho.[32]

In 1920, James Montgomerie,[34] in England, made a number of very careful experiments for the Committee of Lloyds Register, in order to ascertain the elastic behavior of riveted joints. He investigated lap-joints only and the measurements with a very sensitive strain-gauge confirmed the fact that the distribution of both load and stresses was nowhere uniform throughout any of the joints. In the same year, D. Rühl,[35] in Germany, made very careful measurements of the strains in a plate around the rivet holes and showed that the elongation is little influenced by the uneven distribution of stresses, a fact already shown by Batho.[32]

In 1920, Cl. Findeisen,[36] in Germany, also made experiments on the distribution of the stresses in the cover-plates of butt-jointed flat bars, using cylindrical well-fitting bolts as connectors. His investigations and careful measurements are of value in so far as they permit the determination of the distribution of load over the various pins and the checking of certain constants, as explained later.

John S. Watts,[37] in the United States, pointed out in 1921 again, as did Mr. Blackburn,[31] that unsymmetrical double-butt strap joints with wider inner than outer butt strap (as used in boilers) are less efficient than symmetrical double-butt strap joints due to the better distribution of the load in the latter.

In 1922, A. Hertwig,[38] in Germany, proposed using the deflection of the rivet heads as a means of determining the distribution of load on the various rivets. R. Maillard,[39] in Switzerland, dealt in 1923 with the same problem and recommended short joints, rather broader than those used at present, or joints with plates tapered or stepped in the measure in which the forces transmitted decrease, or increase, respectively.

James Montgomerie,[40] in England, supplemented in 1923 his former experiments for Lloyds Committee[33] by tests on lap-joints of heavier plates and larger rivet diameters. His results were similar to those obtained by him for his first series of experiments.

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[33] *Oesterreische Wochenschrift für den öffentlichen Baudienst,* 1919, Nos. 7-8.

[34] *Transactions,* Inst. of Engrs. and Shipbuilders in Scotland, Vol. 63 (March 30, 1920), pp. 284-312.

[35] *Forschungsarbeiten* des Vereins Deutscher Ingenieure, Heft 221, 1920.

[36] *Forschungsarbeiten* des Vereins Deutscher Ingenieure, Heft 221, 1920.

[37] *Boiler Maker,* Vol. 21 (October, 1921), pp. 278-279.

[38] *Der Bauingenieur,* Vol. 3 (March 21, 1922), p. 170.

[39] *Schweizerische Bauzeitung,* Vol. 82 (July 28, 1923), pp. 43-45.

[40] *Transactions,* Inst. of Naval Archts. (London), Vol. 65 (March 23, 1923), pp. 179-197.

In 1923, Friedrich Bleich[41] published in Germany an investigation on the distribution of the stresses in a bar of rectangular cross-section. His study is remarkable in that it is not based, as were all the others, on the usual simplified Navier theory of stress distribution, but on the much more general theory of elasticity for a two-dimensional state of stress, a state to which riveted joints closely conform. He investigated the problem of stress distribution due to local loads, such as are produced by the rivets on a plate, and *vice versa,* and derived, by means of the Airy stress function, formulas for the distribution of the longitudinal stresses in riveted joints.

In the following year (1924) Bleich published another theory.[42] Due to his former study being based on theoretical deductions only, he based his new theory on the work of Arnovļevič,[27] but treated the subject in a more practical manner by introducing for the movement of the plates at the rivets a proportionality factor which he calls the "slip modulus," assuming that the displacements are proportional to the loads transmitted by the rivets. He determined this slip modulus from the experiment by Findeisen[36] for pins, and for rivets from tests by Rudeloff,[28] introducing the value thus obtained directly from experiments into his formulas. By means of this slip modulus he found the percentages of the load transmitted by the various rivets which percentages, however, are nowhere near those determined by either Arnovļevič or Batho,[33] and indicate a much more even distribution of load (his values being only a small percentage greater than those determined from the theory of average load). The slip modulus is the reciprocal value of Mr. Hrennikoff's constant, *k.* Thus, Bleich has already done what the author now recommends in his paper, in that he actually determined the all important constant, *k,* from experimental data, in order to arrive at more correct results.

The Chief Engineer for the Swiss Association of Boiler Proprietors, E. Höhn,[43] carried out in Switzerland in 1925 an exceedingly careful series of experiments on the behavior of riveted joints. Höhn measured the relative displacements between the bars and butt straps of double-butt strap joints for thin as well as for thick plates at various points of the joint. The stress distribution obtained therefrom was found to be nowhere uniform either in the transverse or longitudinal direction of the bar. The strain curves plotted by Mr. Hrennikoff are somewhat similar to those plotted by Höhn, but Höhn's curves are by no means straight lines. Höhn discussed his important results at great length and gave also rules regarding the lay-out of riveted joints.

A further remarkable series of experiments was carried out in 1926 by Commander E. L. Gayhart, U. S. N. Some of the results were published by him and Professor William Hovgaard.[44] This series of experiments is of the utmost importance, not only because they were carried out with exceptionally large double-butt strap joints, but also on account of the great accuracy of the measurements. The results confirmed the findings of the previous experimenters and investigators in that they show for all stages of

[41] *Der Bauingenieur,* Vol. 4 (May 15, 1923), pp. 225-299; May 31, 1923, pp. 304-307; and June 15, 1923, pp. 327-331.

[42] "Theorie und Berechnung der eisernen Brücken," Berlin, Julius Springer, 1925.

[43] "Nieten und Schweissen der Dampfkessel," Berlin, Julius Springer, 1925.

[44] *Transactions,* Soc. of Naval Archts. and Marine Engrs., Vol. 34 (November 11, 1926), pp. 55-74.

loading (just as in Batho's case[32]) that the outer rivets carry by far the greater amount of load, although the inner rivets take an increasing share of it as the load increases. Unfortunately, Commander Gayhart, contrary to Batho, has not published the necessary data of the strain measurement which would permit a check. It is, therefore, not possible to derive from the results of Commander Gayhart the value of the constant, k, of Mr. Hrennikoff.

A. Hertwig and H. Petermann[45] in Germany attempted, in 1929, to solve the same problem by the method suggested by Hertwig,[38] which consisted in measuring by optical means the angular deflections of the ends of the rivet heads due to the bending of the rivets. This attempt was unsuccessful, however, and the experimenters substituted well fitting pins for the rivets just as were used by Rühl[35] and Findeisen.[36] The results thus obtained again showed that the distribution of the load is not uniform. As a check the second of the investigators derived general formulas for this case.

How important the determination of the load distribution in riveted joints is held in England may be seen from the First Report of the Steel Structure Research Committee of the Department of Scientific and Industrial Research of the British Government,[46] published in 1931. Professor Batho's method[32] of calculating the load distribution in riveted joints was incorporated in this report.

Otto Graf in Germany published[47] in 1931 a booklet in which he reported on experiments with riveted and welded bars of various steels under a high permanent load (long-time strength tests) and under tension loads varying between different values. He found that the bearing stresses at the holes changed the state of stress distribution in the main bars so that fracture under repeated loads took place ahead of the outermost rivets of the joint and not at the weakest cross-section in the first rivet row as in ordinary tension tests. He also carried out investigations into the load distribution under varying tensile loads. While these tests were not completed at the time the booklet was published, the results obtained to that time indicated that the distribution of load under often repeated load variations is much more uniform than was to be expected from the older investigations with pins (obviously, Findeisen's investigation[36] and those by Hertwig and Petermann[45] are meant), which latter investigations were carried out as ordinary tensile tests. The increase in temperature of the rivet heads in joints with three rivets in line differed only slightly, thus showing a more even distribution under varying loads.

The results of these tests were published[48] in 1932. The author discussed the importance of the frictional resistance and referred to Wellinger's "Dissertation"[49] in which the means of obtaining the highest possible clamping forces with rivets were described. Not until the frictional resistance is overcome, do the rivets touch the walls of the rivet holes.

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[45] *Der Stahlbau*, Vol. 2 (December 13, 1929), pp. 289-298.

[46] H. M. Stationery Office (Lond.), 1931, pp. 100-179.

[47] "Dauerfestigkeit von Stählen mit Walzhaut ohne und mit Bohrung von Niet-und Schweissverbindungen," by O. Graf, pub. by the V. D. I.-Verlag, Berlin, 1931.

[48] *Zeitschrift* des Vereins Deutscher Ingenieure, Vol. 76, April 30, 1932, pp. 438-442.

[49] Dissertations pub. by the Technical Coll. at Stuttgart, 1931.

Graf discusses here in greater detail than in his booklet the fact that, under the influence of friction and repeated loadings, fracture occurred in the plate slightly ahead of the first rivet. He also showed that the relative displacements of the bars are permanent to a large extent, that a varying load is distributed more evenly than a mere statical load, and that the total deformations were greater than for ordinary tensile tests, the permanent part having increased while the elastic part was altered only slightly.

By investigating the deformation and final destruction of rivets under often repeated loads, Graf made a further considerable step forward. The extent to which the forces in the joint are transmitted by the various rivets coming into direct contact with the wall of the holes, after overcoming the frictional resistance, can be derived from the deformation of the rivets. These were studied by R. Wörnle[49] who, under Graf's guidance, measured the deflection by means of small pins having three collars and being fitted tightly into holes drilled longitudinally through the rivets. The bending of these pins was measured by means of mirrors, and the load transmitted by each rivet was determined therefrom. In this way Graf found that in riveted joints with high rivet clamping forces a very much smaller part of the load is transmitted by the direct action of the rivets than is obtained from the usual calculations which neglect friction. Further, the differences in the load transmitted, under the condition of Graf's experiments, by three or more rivets in line, were less than those in the case where no frictional resistance is assumed to be present.

Attention should further be drawn here to the painstaking investigations of Th. Wyss,[50] in Germany, who in 1923 derived from innumerable strain measurements the distribution of stress in riveted connections, for example, in the gusset-plates connecting members to a chord girder; and, further, to the investigations of Findeisen,[51] in Germany, and Stefan Gállik,[52] in Austria, both published in 1928, who determined the bearing pressure between rivets and plates.

After thus having given a bird's-eye view of the most important research carried out prior to the publication of Mr. Hrennikoff's paper, and after having clearly shown that this problem is not a new one, it is pertinent to investigate the second question, as to whether Mr. Hrennikoff's paper brings a new or simpler method for the determination of the load distribution.

The title of the author's paper leads one to believe that he intends to investigate the work of riveted joints, that is, the storage of energy in riveted joints and the behavior of the joint during such storing of energy; but, in actual fact, all he undertakes is an investigation of the static distribution of load over the rivets and, therefore, it would be more appropriate if he had referred to "the behavior of riveted joints" or "the load distribution in riveted joints."

[50] *Forschungsarbeiten* des Vereins Deutscher Ingenieure, Heft 262, 1923.

[51] Rept., Second Inter. Cong. for Bridges and Structural Eng., Vienna, 1928, pp. 347-364.

[52] Rept., Second Inter. Cong. for Bridges and Structural Eng., Vienna, 1928, pp. 365-402.

Mr. Hrennikoff has chosen for his analysis a "logical" rather than a rigid mathematical treatment. In general, he pictures the phenomena occurring in a riveted joint correctly. It is difficult, however, to agree with his statement that "designing on the basis of friction seems to be quite correct in the case of only one rivet in a longitudinal row * * *." (Mr. Hrennikoff uses "longitudinal row" instead of the accepted terminology which designates rivets in the direction of the transmitted force as rivets "in line" and those transverse to it as "rivets in rows" or "rivet rows"). Even if there were but one rivet, slip at the faying surfaces must occur, because the force, which is transmitted by the rivet due to friction, at the rivet heads and between the plates, is not transmitted at one single point only, but is spread over a fairly wide area. Therefore, it is gradually transmitted from one plate to the adjoining one, thus causing in both plates elongations and distortions that differ according to the magnitude of the force that has already been transmitted up to the point under consideration.

His further statement that "it is reasonable to assume that the other part, X_{AS}, of the total force, X_A, developed by the rivet (that due to shear), is proportional to the slip of plates at Point A," forms the basis of his investigation and, therefore, should be given due prominence. His assumption, however, is no more than a first approximation since the plates do not deform in the longitudinal direction only, but also in the transverse direction, particularly around the rivet holes. On account of this deformation the slip changes over a wide area, and it is impossible to state, the slip of which point should be regarded as "the slip." Thus, Mr. Hrennikoff can undoubtedly refer to the "mean slip" only and thus arrive also at a first approximation only. Arnovljevič[27] and Bleich,[41] who have done exactly the same, have not neglected, however, to state so expressly. Furthermore, the author has completely neglected friction.

Since it is doubtful whether a rigid analysis of the entire problem by means of the theory of elasticity will ever be possible, the writer agrees that some such simplifying assumption is necessary; but a suitable factor should be included allowing for the adaptation of the results of the mathematical analysis to the results from experiments. Mr. Hrennikoff, like Arnovljevič and Bleich, has incorporated this factor in his rivet factor, k.

The derivation of his formulas follows the "logic of Part I," but it must be stated that his "logical" method rather clouds the issue. A plain mathematical analysis accompanied by a simple diagram showing the deformations would have obviated a large part of the complicated description and would have been more to the point in that the main formulas could have been derived directly and would have led to general formulas instead to specific ones. Besides, it would also have obviated the omission of the deductions for cases having more than three rivets and for butt-joints.

If one analyzes mathematically what lies behind the long explanations one finds that Mr. Hrennikoff compares the respective motions of the two plates at the various rivets with one another. For this purpose he chooses as "a fixed point in space" the center line of the rivet at the extreme right-hand end of the upper plate; but the lower plate at this very point still moves to

the right for a distance, $-l$ (in Figs. 3 and 4, this distance has been designated by a).

All this would have been unnecessary if he had drawn a diagram, such as Fig. 16. It would have been evident, therefrom, that the only requirement is to equate the distances of the respective center lines from any rivet head

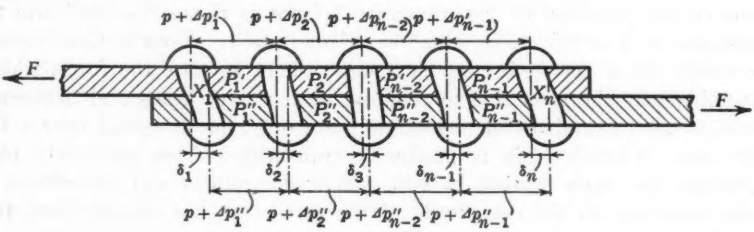

FIG 16

at one plate to the next following rivet head at the other plate, using one time the deformations of one plate and rivet and the other time those of the other plate and rivet. This, after all, is exactly what Mr. Hrennikoff has done without expressly stating it. In this manner, he would have obtained the following system of equations, using the notations of Fig. 16:

$$p + \Delta p'_1 + \delta_2 = p + \Delta p''_1 + \delta_1 \quad \ldots\ldots\ldots\ldots\ldots (28)$$

or,

$$\left.\begin{aligned} \Delta p'_1 + \delta_2 &= \Delta p''_1 + \delta_1 \\ \Delta p'_2 + \delta_3 &= \Delta p''_2 + \delta_2 \\ \Delta p'_3 + \delta_4 &= \Delta p''_3 + \delta_3 \\ &\ldots\ldots\ldots\ldots \\ \Delta p'_{n-1} + \delta_n &= \Delta p''_{n-1} + \delta_{n-1} \end{aligned}\right\} \quad \ldots\ldots\ldots\ldots (29)$$

The respective elongations, $\Delta p'_1$, $\Delta p''_1$, etc., as well as the deflections of the rivets, $\delta_1, \delta_2, \ldots, \delta_n$, can then be expressed by the forces acting in the various plate sections and by the forces transmitted by the various rivets. If the forces transmitted by the various rivets are $X_1, X_2, X_3, \ldots X_n$, and the forces acting in the plates between rivets, m and $m + 1$, are in the upper plate, P'_m, and in the lower plate, P''_m, then there exists the following system of equations (designating the load on the joint by F):

$$\left.\begin{aligned} F - P'_1 &= X_1 \\ P'_1 - P'_2 &= X_2 \\ P'_2 - P'_3 &= X_3 \\ &\ldots\ldots\ldots\ldots \\ P_{n-1} &= X_n \end{aligned}\right\} \quad \ldots\ldots\ldots\ldots (30)$$

Furthermore,

$$\left.\begin{array}{l} P'_1 + P''_1 = F \\ P'_2 + P''_2 = F \\ \dots\dots\dots\dots\dots \\ P'_{n-1} + P''_{n-1} = F \end{array}\right\} \dots\dots\dots\dots\dots\dots\dots\dots (31)$$

and, by addition,

$$X_1 + X_2 + \dots + X_n = F \dots\dots\dots\dots\dots\dots\dots\dots (32)$$

It should be emphasized that Equation (32) is not an independent one. Since,

$$\left.\begin{array}{l} \Delta p'_1 = \dfrac{P'_1 p}{E w t_1};\ \Delta p''_1 = \dfrac{P''_1 p}{E w t_2} \\ \Delta p'_2 = \dfrac{P'_2 p}{E w t_1};\ \Delta p'_2 = \dfrac{P''_2 p}{E w t_2} \\ \dots\dots\dots\dots\dots\dots\dots\dots \\ \Delta p'_{n-1} = \dfrac{P'_{n-1} p}{E w t_1};\ \Delta p''_{n-1} = \dfrac{P''_{n-1} p}{E w t} \end{array}\right\} \dots\dots\dots\dots (33)$$

and, since,

$$\left.\begin{array}{l} \delta_1 = k X_1 \\ \delta_2 = k X_2 \\ \dots\dots\dots \\ \delta_n = k X_n \end{array}\right\} \dots\dots\dots\dots\dots\dots\dots\dots (34)$$

there are six systems of equations in all, $6n$-4 equations for $6n$-4 unknowns—from which the $6n$-4 unknowns (and consequently the unknown rivet forces, $X_1, X_2 \dots X_n$), can be determined. In this way general formulas may be derived which yield for the cases dealt with by Mr. Hrennikoff exactly the same formulas as found by him. It would be very interesting to know, exactly how he has derived the formulas for more than three rivets, because he gives the derivation of this simplest case only; but by whatever mathematical or "logical" process he has arrived at these solutions, the general method is exactly the same as that used by both Arnovļevič[27] and Bleich.[41]

The most important point in the entire evaluation of the formulas, however, is the determination of the factor, k. It is creditable that Mr. Hrennikoff, like Arnovļevič, has attempted to determine this factor analytically. Unfortunately, Mr. Hrennikoff has not given this derivation so that it is impossible to determine the accuracy of his results. Mr. Hrennikoff obviously feels, and rightly so, that this determination can only be regarded as a first step and advocates the determination of k by experiment. Bleich has previously done this by attempting to determine this value, or rather its reciprocal value, from Rudeloff's experiments.[28] Obviously, Bleich did not know of Batho's anaylsis and experimental data,[32] because he could have used these to check his value for the "slip modulus." Thus, Bleich went a step further than Mr. Hrennikoff, and the latter's results can be of no more than academic interest.

The writer would like to give Mr. Hrennikoff credit for one advance that his analysis has established. He has resolved the cross-sectional area of the plates and cover-plates into width and thickness. Thus, he is able to get, for the same plate width, from his formulas directly the rivet coefficient for any plate thickness while the formulas of his predecessors do not allow of this directly, requiring a further though short calculation.

In valuing Mr. Hrennikoff's paper it must be pointed out that he has given a solution for a limited number of cases only; that is, for lap-joints containing up to six rivets and for butt-joints containing up to three rivets. His predecessors, however, have given general formulas which they evaluated for various cases, for double-butt strap joints up to six rivets. If one further compares Mr. Hrennikoff's four conclusions with those given by his predecessors, it is difficult to find anything new in them.

It may not be amiss to state further, that Mr. Hrennikoff has not taken into consideration the bending in lap-joints due to the moment created by the plates not lying in one plane. This changes the distribution of forces materially on account of the bending it induces in the rivets besides that present if the plates were not subjected to this bending moment.

At the beginning of Part I, Mr. Hrennikoff makes the statement that riveted joints "are not well understood." To this very true statement the writer would like to add that "the literature on riveted joints" does not seem to be well known, either. This has been the reason why he has prepared[53] a rather comprehensive "Bibliography on Riveted Joints." He would recommend its careful study to all who intend to carry out any experimental or theoretical work on riveted joints. It is simply amazing how much has been duplicated, how many immensely costly experiments have been carried out, the readings and data of which have never been published and, therefore, cannot be evaluated to the limit of their usefulness. Any progress in riveted joints can only come from a careful study of all the results obtained thus far and from outlining further tests with the idea of checking up experimentally the factors involved, a procedure which has also been recommended by Mr. Hrennikoff.

The writer would like to take this opportunity to sound a note of warning with regard to the calculation of riveted joints based on insufficient assumptions. As almost no structure is subjected to a steady direct load, but rather always to variations in load, these latter should receive due attention. Further, it is useless to try building up a rational theory of riveted joints without taking the frictional resistance of the joint into consideration. Only by the co-ordination of all factors having a bearing on the resistance of riveted joints can a theory be arrived at which will describe the behavior of riveted joints more or less correctly. Graf's investigations[47,48] form a first step toward this end. If Mr. Hrennikoff's paper will have the effect of starting a new movement in the direction outlined by the writer, the latter can do no better than to thank the author heartily for his paper.

[53] "Bibliography on Riveted Joints" by A. E. R. de Jonge, *Research Publications*, A. S. M. E. (1933).

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852
DISCUSSIONS

STUDY OF STILLING-BASIN DESIGN

Discussion

By Messrs. E. W. Lane, C. C. Inglis, and F. Knapp

E. W. Lane,[4] M. Am. Soc. C. E. (by letter),[4a]—The best form of protection against scour at the toe of an overflow dam depends on the relative elevation, over the full range of discharge anticipated, of the natural tail-water level below the dam and the water-surface elevation required to form the hydraulic jump on a horizontal apron at the stream-bed level. When the tail-water level is below that required to form the jump, the best form of protection often is to increase the depth of the tail-water, either by increasing the water depth by excavation, or by raising the tail-water level by a secondary dam. For the latter case, if the flow over the secondary weir is not effected by the tail-water level down stream, the data contained in Mr. Stanley's paper are of great value to the designing engineer, and the profession is indebted to him for making it available. Since the tail-water level can rise above the crest of the secondary dam—if ogee in shape, two-thirds as high as the head-water—without materially influencing the discharge, the data will apply directly to a large range of conditions.

An extensive series of model tests similar to those of Mr. Stanley was made in 1930-31 for the design of the spillway of the Cle Elum Dam of the U. S. Bureau of Reclamation. This spillway was designed for a maximum discharge of 40 000 sec-ft and a water surface drop of 110 ft. It was of the trough type, controlled at the upper end by gates. The width at the upper end was 200 ft. At the middle this was narrowed to 100 ft and expanded to 200 ft at the lower end, where it extended the stilling pool. The trough was 900 ft long, and its shape as well as that of the stilling pool was developed by model tests. As there was no rock within reasonable depth, the entire structure was founded on gravel. The height of the tail-water was insufficient to form the hydraulic jump on an apron at stream-bed level, but instead of building a secondary weir to secure the depth necessary for jump formation it was obtained by depressing the pool bottom 11 ft below the river-channel bottom.

Note.—The paper by C. Maxwell Stanley, Jun. Am. Soc. C. E., was published in November, 1933, *Proceedings*. This discussion is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

[4] Research Engr., U. S. Bureau of Reclamation, Denver, Colo.

[4a] Received by the Secretary January 31, 1933.

An extensive series of tests was undertaken to determine the best form of stilling basin. A model of the spillway on a 1:50 scale was set up in the hydraulic laboratory of the Colorado Agricultural College, at Fort Collins, Colo. The effectiveness of the various forms of stilling pool was determined by the scouring effects in a bin in which sand was formed to simulate the river bed down stream from the pool. Preliminary tests showed that duplicate runs would produce the same scour results, that the amount of compacting of the sand was of little importance, and that a run of 1 hr was sufficient to secure practically the same result as a flood of great duration in the spillway.

The first design of the stilling pool provided a horizontal floor, 102 ft long, down stream from which was a section 44 ft long sloping upward 11 ft to river-bottom level, followed by a 10-ft level floor. The tests showed considerable scour at the end of this floor, which could not be corrected by sills. A steeper upward sloping section produced even less desirable results. Much better success was obtained by making the floor level throughout, 11 ft below river-bed level, and permitting the rise from the pool floor to the river-bed level to be formed in the river bed down stream from the pool.

Although fairly satisfactory results were secured with a simple pool with a level floor, the scour was less if some form of sill was placed at the down-stream end of the floor. These sills were in no sense secondary weirs, as were those used in the author's experiments, since they did not control, or even materially influence, the tail-water level. The height of most of them was only about 30% of the water depth. Their function was to form an eddy behind them, at the down-stream edge of the floor in which the current in contact with the river bed moved up stream and dragged bed material up toward the end of the floor, rather than scouring it away.

A number of types of sill were used. The one on which most of the tests were made was the dentated type introduced by Dr. Theodor Rehbock.[5] Another had an ordinary ogee dam section, and a third was trapezoidal in section with a 45° slope on its up-stream side and with a vertical down-stream face. All these sills were located at the down-stream edge of the apron. Most of them had a height of 10 ft. Tests were made, however, which showed that greater scour occurred with smaller heights and less with greater heights. The 10-ft height was used because the volumes of higher weirs, particularly in the case of the dentated sill, became excessive for great heights. All these forms gave good results, those obtained with the dentated sill being slightly better than the others, and the dentated form was adopted for the structure.

Tests were made to determine the best slope for the floor entering the stilling pool. Slopes of 1 on 1.5, 1 on 2, 1 on 3, and 1 on 4, were tested. In most cases the scour increased as the slope became steeper; it is believed that this was due to the fact that the hydraulic jump forms more perfectly with the flatter slopes. With the dentated sill the scour was practically the same at all slopes and since other considerations favored steeper slopes, the 1 on 1.5 slope was adopted for the structure. Tests also showed that

[5] *Transactions*, Am. Soc. C. E., Vol. 93 (1929), p. 527.

with a curved surface between the sloping incoming floor and the level pool bottom less scour was produced than when there was no such transition. This also is believed to be due to the fact that the curved surface tended toward the formation of a more perfect jump.

In order to secure the greatest economy in the construction of the pool, extensive experiments were performed to determine the best combination of width, depth, and length. Experiments were made with pool widths corresponding to 200, 150, and 120 ft. For each width three or four pool depths were used and with each depth, several lengths were tested. In all, thirty-eight conditions were investigated. The 10-ft dentated sill was used in all tests and the discharge and tail-water level was that corresponding to the maximum expected flood. The scour of the bed down stream from the floor was slight in all cases, but the action on the banks differed considerably. The results were unusually consistent, which gave considerable confidence in them.

A comparison of the tests is given on Fig. 14, in which the various widths, depths, and bottom elevations are shown, and lines representing equally severe scour on the channel sides are drawn. The tests numbers are in circles, and

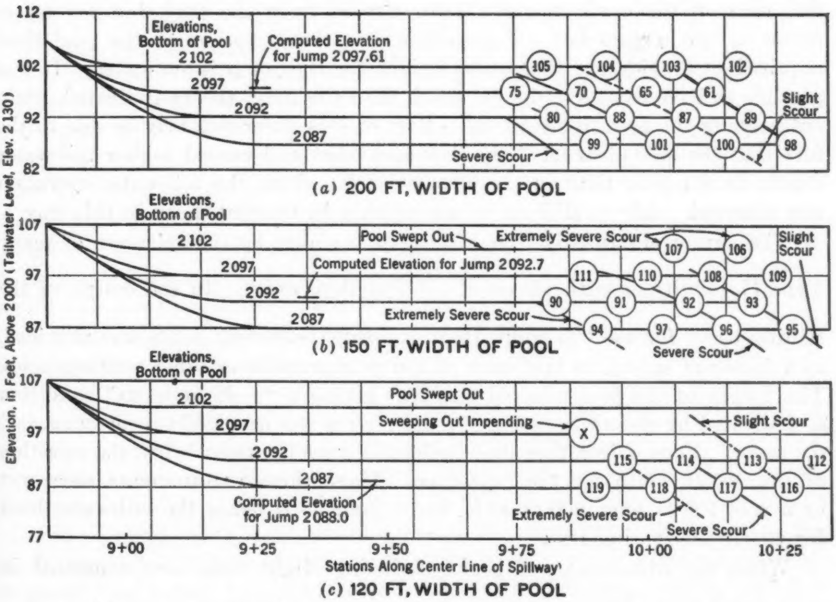

FIG. 14—COMPARATIVE SCOUR WITH POOLS OF VARIOUS LENGTHS, DEPTHS, AND WIDTHS

the location of the circles indicates the end of the pool floor in that test. The results indicate that the severity of scour increased as the pool length was decreased, but that the scour was not decreased by deepening the pool. The former result might be reasonably expected, but a decrease of scour with

increased depth seemed probable, due to the greater volume of water in the pool with the greater depth and, therefore, a greater dissipation of the energy.

The results showed that for the same pool length, the severity of the scour was nearly independent of the depth as long as the jump remained in the stilling basin. This action is believed to be due to the more efficient formation of the jump with the smaller tail-water depth. With high tail-water levels the jump forms on the sloping floor leading into the pool. In such cases, the incoming water tends to dive under the water in the pool without causing a well-formed jump. As the tail-water level is lowered, the jump forms farther down the slope where the bottom is more nearly level; the tendency to dive under is less, and a more perfect jump is formed. This tendency to form a more perfect jump at smaller depths offsets the advantage in the greater depths, of the greater mass of water in which to dissipate the energy, with the result that the efficiency of the basin is independent of the depth, as long as the jump remains in the pool.

If the floor was raised too high, the depth was insufficient to cause the hydraulic jump in the stilling basin and the pool was swept out, as in Runs Nos. 106 and 107 (Fig. 14(*b*)). Such conditions produced severe scour on the sides of the earth channel down stream from the pool, but no severe scour on the stream bed. For each width, the elevation of the pool floor required to provide the theoretical height for the jump is indicated. It was possible to have considerably less depth than the jump theory indicated, without causing the pool to sweep out. Part of this difference may be due to the fact that the sills obstructed the flow somewhat and caused higher tail-water depths in the pool than in the stream below, where the tail-water elevation was observed. All the difference can scarcely be accounted for in this way.

The author obtained similar results, as is shown by the existence of many Type II conditions with values of $\frac{D_w}{D_j}$ less than unity. In the design of the stilling basin for the Cle Elum Dam, the full theoretical depth was provided, as a factor of safety, to take care of bed retrogression or other contingencies. The length of the pool adopted was that giving a "a slight scour" condition as indicated by the author's Fig. 1, and rip-rap was provided to protect against the scour. Economic rather than hydraulic considerations led to the selection of the 200-ft width for the prototype. These three requirements were met by a pool 100 ft long, 200 ft wide, and a floor 33 ft below the tail-water level for 40 000 sec-ft discharge.

When the dimensions of pools producing slight scour are expressed in terms of $\frac{D_w}{D_j}$ and $\frac{L}{D_w}$, and plotted as in the author's Figs. 7, 12, and 13, they indicate lines roughly parallel to the inclined portion of the solid line on Fig. 7, which divides the satisfactory from the unsatisfactory cases, but they give lower values of $\frac{L}{D_w}$. For the experiments with the 200-ft bottom, the

corresponding values of $\frac{L}{D_w}$ are only two-thirds as great as the author's Series *A*. For the experiments with the 150-ft and 120-ft widths they are even less, being only 58% for the latter width. There are several factors which may account for this difference. One is that the incoming slope for the Cle Elum experiments was 1 on $1\frac{1}{2}$ as compared with 1 on $\frac{1}{2}$ (approximately) in the author's experiments. The flatter slope would probably give a more efficient hydraulic jump. Furthermore, on the flatter slope, the jump would begin farther up stream from the point where the pool was assumed to begin, which was at the intersection of the incoming slope with the pool floor, and, therefore, for a given nominal pool length, the effective length would be greater for the flatter slope. A third factor is that in the Cle Elum type the pool may continue in effect beyond the end of the floor, as the sill does not obstruct the flow, but only protects the bed from scour.

In order to make the results of the Cle Elum tests as readily applicable as possible to other conditions, Fig. 15 has been prepared. It is based on the assumption that the distribution of discharge in the experiments was uniform across the entire pool, and on the theory of model similitude. The foregoing assumption is believed to be substantially true. Suppose, for example, it is desired to design a spillway for a discharge of 25 000 sec-ft and a fall of 80 ft (plus friction loss) from reservoir level to tail-water level. The velocity head of the water entering the stilling basin, at the elevation of the tail-water, would then be 80 ft. Entering Fig. 15 with this value gives a model ratio of 43. This means that if the model experimented on had been considered to be a 1:43 model (instead of a 1:50 model as was required, in order that it meet the conditions for the Cle Elum Dam), the proportions would be correct for a velocity head of 80 ft.

Since, by the model theory, the form tested would represent a 1:43 scale as accurately as the 1:50 scale, the model would represent reliably the desired conditions as well as those at Cle Elum Dam. With a model ratio of 43, the lines representing discharge per foot of pool width show 135, 170, and 220 sec-ft per ft of width as determined from the experiments with model pool widths of 4.0, 3.0, and 2.4 ft, respectively. For a discharge of 25 000 sec-ft, a pool 114 ft wide would be required if the discharge was 220 sec-ft per ft of width as determined from the 2.4-ft model pool. The line representing depth of pool tail-water level based on the 2.4-ft pool width shows for a 1:43 model, a depth of pool below tail-water level of 36 ft, and the similar line for length of pool required, for a 1:43 model gives 99 ft. Since the sill height of the model was 0.2 ft, that required would be $0.2 \times 43 = 8.6$ ft high. Thus, Fig. 15 shows that a pool, 114 ft wide, 99 ft long, and 36 ft deep, with a sill 8.6 ft high, would meet the required conditions.

Other pool dimensions equally suitable, could be obtained from the lines representing the results determined from the 3.0 and 4.0-ft model pools. Of these three sets of dimensions, the most advantageous from the standpoint of cost or other considerations, may be chosen. Care should be exercised, however, to use throughout the dimensions determined from the lines based

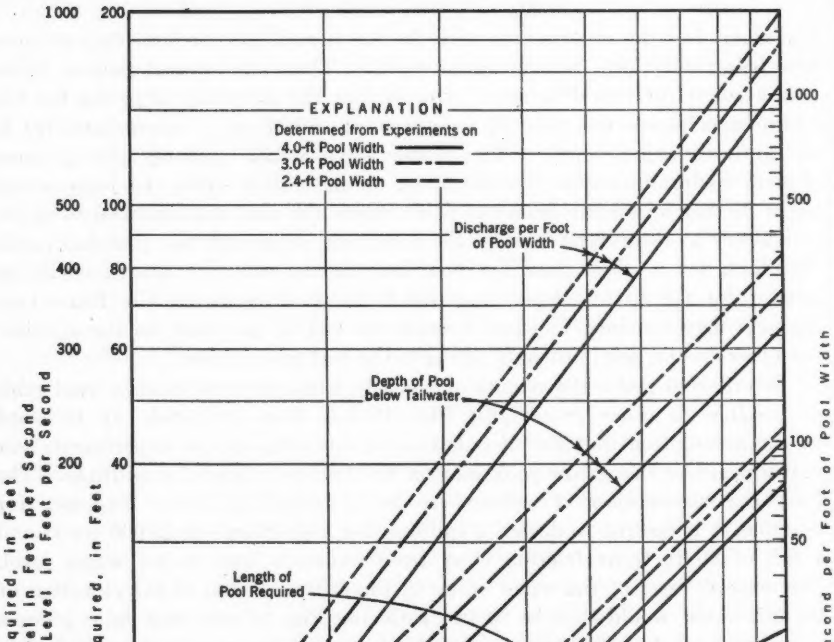

FIG. 15.—DIMENSIONS REQUIRED FOR STILLING POOLS AS SHOWN BY RESULTS OF MODEL TESTS

on the same model pool width, and not to use, for example, the depth obtained from the 4.0-ft model pool and the length as indicated by the 3.0-ft pool. It is recognized that the dimensions indicated by Fig. 15 may not be very practical in certain cases. This simply means that the Cle Elum model tests are not applicable to those cases, and data must be sought elsewhere for their design.

In applying this diagram the following conditions to which it is applicable should be kept in mind: The flow must be equally distributed across the stilling pool and in the direction of its axis. The tail-water rating curve at the site must be such that ample pool depth is available at all discharges to cause the jump to form in the basin. The top of the sill must be sufficiently below the channel bottom so that a drop of the water level will not result at any discharge from the obstruction caused by the sill. There will be considerable wave wash on the sides down stream from the pool, and riprap protection will be required, especially if the pool is intended to be used over long periods at flows approaching its capacity.

Although the experiments were performed with a dentated sill it is believed that the ogee or beveled sill types could be used with slightly less desirable results. The experiments were performed with a slope of 1 on 1.5 entering the pool and a transition curve at the bottom of the slope. It is believed that flatter slopes with the transition curve would produce equally as good results, but that the effects of steeper slopes might be less desirable. The writer agrees heartily with Mr. Stanley that "the day is not far distant when experiments with models will be a necessary part of the design of every overflow dam." The data given herein, therefore, should not be considered as removing the necessity of model tests, but only as the basis for preliminary designs, to aid in narrowing the field to be covered by the model studies, and for the design of unimportant structures of insufficient magnitude to bear the expense of model tests.

C. C. INGLIS,[6] ESQ. (by letter).[6a]—For the very narrow limits of the experiments—namely, two-dimensional parallel flow, horizontal bed, and free flowaway down stream—stilling basins of Types I and II give highly satisfactory results; but this is only a special case of a very great problem. More generally, dissipation of energy is a three-dimensional problem, and eddies with vertical or complex axes cause much damage. Where energy has to be dissipated and eddies have to be prevented in canals, a combination of a "baffle" and a "deflector" has given satisfactory results in Sind.[7]

The baffle is made equal in height to the free-flow depth and is constructed at a distance from the toe equal to $5\frac{1}{4}$ times the free flow depths. The baffle is fixed at such a level that, with the natural down-stream water level, the wave will form at the toe of the fall—under which conditions the standing wave dissipates the maximum amount of energy. The deflector is fixed at

[6] Superintending Engr., Irrig. Development and Research Circle, Poona, India.

[6a] Received by the Secretary February 8, 1933.

[7] "The Dissipation of Engery Below Falls," by C. C. Inglis and D. V. Joglekar, *Technical Paper No. 44*, Public Works Dept., Bombay, India.

the end of the pavement, and is equal in height to one-twelfth the maximum depth of water. For such conditions Types I and II gave highly unsatisfactory results.

F. KNAPP,[8] Esq. (by letter).[8a]—The author deserves credit for the experiments made to rationalize the design of stilling basins at the toe of an overflow dam. Especially valuable are Figs. 7, 12, and 13, which show the relationships between the fundamental and controlling factors. Mr. Stanley classifies five distinct types of stilling-basin action, Types I and II (the most efficient) being identified as roller and jump action, respectively. The writer does not feel satisfied with the distinction between these types. From the description given under the heading "The Stilling Basin," Type I can also be considered as jump action with the nappe submerged by tail-water. Type II represents free nappe with repelled jump. According to B. A. Bakhmeteff,[9] M. Am. Soc. C. E., the ratio,

$$\frac{\text{Difference of elevation, between head and tail-water}}{\text{Height of dam}}$$

defines the two types of jumps. Type II requires a ratio > 0.75 and Type I, a ratio < 0.75 approximately, the correct value depending on the overflow conditions of the dam. Experimental observations by Bazin confirm Professor Bakhmeteff's deductions.

Under the heading "The Results, Series *A*", Mr. Stanley states that the action classified as Type II occurs when the ratio, $\frac{D_w}{D_j}$, is between 0.85 and 1.15, or substantially equal to unity, and the ratio, $\frac{L}{D_w}$, exceeds about 4.5. In other words, the depth of the pool must be that required for the formation of a hydraulic jump and the length of the pool must not be less than about $4\frac{1}{2}$ times the depth of the jump. There are cases, however, that require a ratio, $\frac{L}{D_j}$, considerably greater than 4.5. Fig. 7 shows this ratio varying from 4.6 to 7.2 for Type II action and a ratio, $\frac{D_w}{D_j}$, equal to unity. With the length of the pool determined, either theoretically or empirically, the problem of designing a stilling pool for Type II action, would be solved. This length may be determined by the following considerations.

The loss of energy in a hydraulic jump is always smaller than the Borda-Carnot loss, $\frac{(V_1 - V_j)^2}{2g}$. A certain distance is required before flying water in the jump can be slowed down to normal, flowing water. Consequently, with the velocities changing gradually, the loss of head produced by the jump must

[8] Designing Engr., The Sao Paulo Tramway, Light & Power Co., Ltd., Sao Paulo, Brazil.

[8a] Received by the Secretary February 17, 1933.

[9] "Hydraulics of Open Channels," *Engineering Societies Monographs*, New York and London, 1932.

be smaller than $\dfrac{(V_1 - V_j)^2}{2g}$. In other words, there must be a relation between the length of the jump, L, and the loss of head in the hydraulic jump, namely $\left(d_1 + \dfrac{V_1^2}{2g}\right) - \left(d_j + \dfrac{V_j^2}{2g}\right)$. Based on published experiments[10], the writer has deduced the following empirical formula (in metric units) for the length of the jump:

$$L = \left(62.6 \frac{D_1}{H_1} + 11.3\right) \left[\frac{(V_1 - V_j)^2}{2g} - (H_1 - H_j)\right] \ldots\ldots\ldots (2)$$

in which, $H = d + \dfrac{V^2}{2g}$.

Unfortunately, it is not possible to present a comparison of this formula with the results obtained by the author. The writer is unable to check the depth, D_1, of the jump from the values of discharge and the depth, D_j, given in Table 1. The writer hopes Mr. Stanley will clarify this point in his closing discussion.

[10] "Untersuchungen über den Wechselsprung," von Safranez, *Der Bauingenieur*, 1929, No. 38, p. 676.

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DISCUSSIONS

A PROBLEM OF SOIL IN TRANSPORTATION IN THE COLORADO RIVER

Discussion

By MESSRS. C. E. GRUNSKY, IVAN E. HOUK, AND H. M. ROUSE

C. E. GRUNSKY,[33] PAST-PRESIDENT, AM. SOC. C. E. (by letter).[33a]—It is with satisfaction that the writer notes a reviving interest in some of the larger problems of the Colorado River. Of this reviving interest, this paper on a method of freeing the river water of a large proportion of its bed load of silt at the head-works of the proposed All-American Canal, gives evidence. The subject is certainly one which merits the careful attention and study of the Engineering Profession. The magnitude of the problem has gone unquestioned and yet, to the present time (1933), how little is known of the physical characteristics of the silt particles, carried by the waters of the Colorado, which are to be handled! What are their dimensions? What shapes predominate? How much colloidal matter is with the silt? It is indeed refreshing to have the author emphasize the magnitude of the bed load of the Colorado. In this matter, however, the writer is not in full agreement with Mr. Rothery. He is willing to concede, and in an earlier paper[34] has endeavored to demonstrate, a large bed load, but found nothing to warrant the author's intimated conclusion (see heading "The Bed Load") that the bed load of the river exceeds its suspended load.

The writer is in full accord, too, with the author's contention that there will be silt problems at points of water diversion below the completed Hoover Dam, even if in the course of time these may be greatly simplified. The writer is, for example, still among those who do not understand the reason why the Metropolitan District did not take advantage of the desilted supply of clear water which will be available at the Hoover Dam, instead of selecting a diversion point for domestic water far below it, where the engineer will have to contend with all the annoyances of a turbid river supply.

The writer does not care to enter upon a discussion of the merits of the author's plan for ridding the water of its bed load at the head of the pro-

NOTE.—The paper by S. L. Rothery, M. Am. Soc. C. E., was published in December, 1932, *Proceedings*. This discussion is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

[33] Cons. Engr. (C. E. Grunsky Co.), San Francisco, Calif.

[33a] Received by the Secretary January 28, 1933.

[34] "Silt Transportation by Sacramento and Colorado Rivers and by the Imperial Canal," by C. E. Grunsky, Past-President, Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. 94 (1930), p. 1124 *et seq.*

posed All-American Canal, but in this connection, would merely say that if the bed load can be eliminated, the problem of canal operation and maintenance becomes relatively simple. This is due to the fact, well established by all the available observations, that Colorado River water, entering a canal from the river, will carry its suspended load practically without change throughout the length of the canal. Any device for removing or, at any rate, for greatly decreasing the bed load, should be simple and relatively inexpensive. The nearer it can be kept in design to a sedimentation basin with suitable sluicing facilities the better.

IVAN E. HOUK,[35] M. AM. SOC. C. E. (by letter).[35a]—The author has contributed a comprehensive, timely and interesting review of the many important problems involved in the measurement and control of silt transportation in the Lower Colorado River.

Many of the data used as a basis for his discussions are of an approximate nature. Consequently, his conclusions can only be considered approximate. Probably the greatest value of his review is the emphasizing of the complexity of the silt problem and of its importance in future river-control work.

It is indeed unfortunate that accurate data are not available to show the average depth of periodic flood scour in the section of the Colorado River under discussion. Isolated observations of scour may be very misleading as regards representative conditions in a 300-mile stretch. River gauging stations are usually located at cross-sections where the stream is more or less contracted in width. Therefore, depths of scour observed at such stations usually tend to be greater than the average rather than smaller. Any computation of bed load based on an estimated average depth of scour is nothing more or less than a pure guess. It may be several hundred per cent. in error in either direction.

The writer believes that the author's guess of a spring flood bed load equal to twice the estimated annual suspended load is much too large, instead of being conservative as the author concludes. It is understood that river-bed excavations at Hoover Dam show that the depth of periodic scour in Black Canyon has been less than was formerly supposed. Actual measurements at other locations might show similar conditions. In order to make a fairly accurate estimate of the average depth of periodic flood scour in the river channel above Yuma, Ariz., it would be desirable to have detailed cross-sectional measurements during flood periods at locations about ten miles apart along the entire length considered, as suggested by Mr. Rothery. Even if satisfactory information were available regarding the average depth of scour, it would not be possible to make an accurate calculation of the total bed load, since a large proportion of the scoured material undoubtedly is carried in suspension. The estimate of total bed load based on an average depth of scour would necessarily have to make proper allowances for the suspension factor.

In rivers flowing through gravel and boulder formations undoubtedly there is a definite boundary between suspended and bed load silt. Such a

[35] Senior Engr., U. S. Bureau of Reclamation, Denver, Colo.

[35a] Received by the Secretary March 10, 1933.

condition may exist, temporarily, in some sections of the Lower Colorado River, at times when the erosion is deep enough to reach coarse gravel or boulder deposits. However, it is probable that during the whole, or, at least, during the greater part of the Lower Colorado River floods, there is a gradually increasing thickness of the mixture of solid particles and water near the bed of the river, rather than a definite line of demarcation. This is indicated by the difficulty in finding a solid bottom while making soundings during flood periods. The writer has experienced such difficulties in gauging floods in rivers flowing through much coarser materials than those characteristic of the Lower Colorado.

In measuring the silt content of the Colorado River flow at Yuma, approximately one-third of the samples were taken as close as practicable to the bottom of the river. Consequently, it is believed that the estimated silt flow based on the Yuma measurements includes a large proportion of the material eroded along the bed of the river at up-stream locations, material considered by the author as making up an unmeasured bed load flow.

The accurate measurement of silt flow is always a difficult matter, whether the material is moving in suspension, by saltation, or as a bed load. The writer cannot agree that where samples are taken near the bottom of the river, with the sampling bottles pointed up stream, the "increased percentages are properly part of the unknown bed load." Holding the bottles in such a position naturally results in catching more solid particles than are actually present in a given volume of flow. Such results cannot properly be used in calculating the silt load of the stream on the basis of a current-meter gauging of discharge. In order to determine the correct proportion of silt present in river flow it is necessary to adopt some method of segregating actual samples of the flow instantaneously at different locations in the cross-section, such as a long cylinder, open at both ends, placed in a direction parallel to the lines of movement and arranged so that the two ends of the cylinder, or the two ends of a central section thereof, can be closed simultaneously. A silt sampler was developed to operate in this manner.[36]

The author is undoubtedly right in his conclusion that "both suspended and bed loads will be present in the curtailed river flows below the Hoover Reservoir." However, his expectation that the suspended loads will "decrease in extent to almost negligible quantities as the regimen of the stream becomes stabilized" may or may not be fulfilled. As soon as Hoover Dam is put into operation, cloudburst occurrence, resulting cloudburst erosion, and details of surface geology on the tributary drainage areas below the dam will become important factors as regards the character and quantity of solid materials transported by the regulated flow. The writer is not familiar with the surface geology of the drainage areas of the Lower Colorado River. If there are large areas of extremely fine, surface silt deposits in the lower tributary areas, and if cloudburst rainfalls happen to fall on such deposits, large quantities of material will certainly be eroded and washed into the main channel and will tend to keep up the turbidity and supply of suspended material.

[36] "Silt in the Colorado River and Its Relation to Irrigation", by Samuel Fortier, M. Am. Soc. C. E., and Harry F. Blaney, Assoc. M. Am. Soc. C. E., *Technical Bulletin 67*, U. S. Dept. of Agriculture, p. 12.

The construction and putting into operation of the Elephant Butte Reservoir in New Mexico changed the soil transportation problem of the Rio Grande at El Paso, Tex., from what was essentially a silt problem to what is now essentially a sand problem, due to the fact that the tributary drainage areas below the dam are covered with sand and gravel deposits rather than with fine silt or clay deposits. The change would have been quite different if the Rio Puerco, of New Mexico, had joined the Rio Grande below Elephant Butte Dam, instead of above it. The Rio Puerco flows through deep and extensive deposits of fine silt. Its flow contains large quantities of silt during normal stages as well as during flood periods. A detailed study of surface geology, cloudburst occurrence, and cloudburst erosion on the tributary drainage areas of Colorado River below Hoover Dam would undoubtedly aid in formulating accurate predictions regarding the character and extent of the silt load to be expected after the reservoir is put into operation.

H. M. ROUSE,[87] ASSOC. M. AM. SOC. C. E. (by letter).[87a]—An opportune and valuable study has been made by the author, indicating the need of drastic changes in the original published plans of the desilting basins for the proposed new Imperial Valley Diversion Works. He emphasizes particularly the large quantity of bed sand present in the river flow, which can best be handled by methods that differ from those used in handling the silt or suspended load.

The bed sand should be diverted neither into the Main Canal nor into desilting basins unless it may be proved by various experimental models of the diverson works that it cannot be separated and caused to flow continuously back into the river. Mr. Rothery's suggested plan for this part of the diversion works is a fine basis for beginning experiments.

The writer realizes from experience on the Imperial Valley Canal System, that bed sand is the most troublesome and costly factor in canal operation and maintenance, but believes that the suspended load, or silt, should also be removed, and that there will be enough of it in the future to warrant the construction of desilting basins. Although ranchers of the present irrigated area do not agree as to the value of the silt on the lands, a large majority believe it has little or no value as a soil renewer, and that it is a detriment to many crops.

The removal of bed sand will decrease greatly the cost of lateral canal cleaning in the present irrigated area. Silt removal will further decrease that cost and will stop the growth in the size of the unsightly banks or levees of the canals caused by cleaning out the silt and sand deposits. Bed-sand removal will allow of the installation and successful operation of underground pipe-line water distribution systems on the ranches, which are not now feasible on account of lack of slope or opportunity for sluicing the pipe lines. Silt removal will allow the use of pipe lines on very light slopes, or on whatever slopes clear water requires.

Possibly bed-sand removal will not require additional structures for slope control in the existing canal system, while silt removal will probably require

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[87] Engr., Colorado River Land Co., Calexico, Calif.

[87a] Received by the Secretary March 17, 1933.

some construction for slope control or against bank erosion, or both. However, this construction will come gradually and, in some cases, will permit of improvements in operating conditions. Furthermore, the slope control may be obtained in many cases at the time of building more diversion structures, which will be required from time to time as the lands are subdivided into smaller units.

The writer believes that the desilting basins are needed, in addition to the desanding works. However, the location for the desilting basins can be chosen where the large areas (which the author rightfully emphasizes are needed) are available without expensive quarrying in the mountains or expensive foundations in the river bed. This points to a location below the proposed diversion works where the Main Canal location begins to swing away from the river; or in the vicinity of Araz Junction, Calif., where the location is again near the river; or between the latter and Pilot Knob, Calif., where the location of the Main Canal can be near enough to the river to afford ample slope for sluicing the basins.

A main canal slope can be used which without altering the basic project scheme, will carry the silt to the desilting basins, and the basins may be designed, constructed, and operated, so as to leave any percentage of silt desired in the irrigation water for the stabilization of canal banks in the sandy mesa lands and for the enrichment of the lands themselves. Should it be decided to leave some silt in the irrigation water for the last-mentioned purposes, additional desilting basins can be built at the western edge of the mesa lands, to clear the water for use in the present irrigated area. The construction of the desilting basins away from the diversion works would allow more freedom in the design of the desanding works.

The construction and testing of models of the diversion works, with desanding and desilting devices, and of models of desilting basins, if not already done, is needed for the final design of the head-works. Model study is needed, especially for the following (referring to Figs. 5 to 8): (*a*) Determination of the minimum necessary distance from the up-stream edge of the horizontal partition, to the irrigation water-control gates; (*b*) thickness of the lip of the horizontal partition and the material to be used in its construction; (*c*) shape and length of the approach channel; (*d*) type and location of drift-handling booms, gratings, or machinery; and (*e*) avoidance of the formation of bars at the mouth of, or in, the approach channel with floods of varying stages and duration, since it is possible that provision of more flood-gates in the dam proper, with less resulting variation of stage during floods, may be found desirable.

The enemy of desanding or desilting, of course, is turbulence. The author refers to "a side offtake without under-sluices" as being an "effective selector of bed silt." The head-gate of one of the main canals of the present canal system is a particularly good example of this. Although supplied with raised flash-board "sill" or overpour control, the water diverted carries its proportion of the bed sands traveling in the Main Canal above the diversion.[38]

[38] Reference is made to Photograph B, Pl. 2, *Technical Bulletin No. 67*, entitled "Silt in the Colorado River and Its Relation to Irrigation," by Samuel Fortier, M. Am. Soc. C. E., and Harry F. Blaney, Assoc. M. Am. Soc. C. E., which shows the bed of the aforementioned canal below the head-gate.

At a point in another of the main canals a structure was built which combined the purposes of a check or control structure in the Main Canal, a head-gate for another canal, and an important waste-gate. The placing of the waste-gates some distance up stream from the other gates and in the bed of the Main Canal, was seriously considered, with the idea of removing a large amount of the traveling bed sands with the water wasted at this point. Practical difficulties prevented such location, and the waste-gates were placed vertically below the gates of the irrigation water control, the waste water entering a tunnel, the roof of which is the floor of the irrigation water-control gates above, the tunnel emptying at the side of the combined structure. Turbulence in front of the structure causes the bed sands to be carried on with the irrigation water as was expected, the percentage of sand in the irrigation water and the waste water being practically the same This is a case of take-off in a direct line with the Main Canal, and with under-sluices provided, although, of course, the under-sluices could only be used to the extent that water was wasted.

One of the troublesome effects of bed sand in the main canals of the present irrigation system is the fluctuation of the discharge at one point, with no changes at the next control point above. This trouble is evidenced by increases and decreases of hundreds of second-feet in the discharge at key waste-gates, the change amounting to a maximum of 700 sec-ft of a main canal flow of, say, 5 000 sec-ft. For some time gauges were read on a stretch of the Main Canal, and these readings indicated a raising of the water surface, because of the bed sands being temporarily deposited; then they indicated a lowering of the water surface, releasing the water which had been temporarily "stored" by the building of the bed-sand "dam" in the bottom of the channel. In some cases, a cycle of one of these fluctuations would take place in 5 or 6 hours and, in other cases, the decrease might be slow and the increase very sudden, the cycle taking 24 hours, or more. Sometimes the increase only would be noted, the decrease presumably being hidden in daily water handling, or in a very slow decrease of some days' duration. In order to cover the great fluctuations caused by the bed sands, large waste-gate capacity and a great quantity of water is required in order to avoid fluctuations of delivery to main canals, laterals, and, thus, to irrigators.

Seasonal movement of bed sands in the Main Canal also causes trouble in the spring, the bed of the canal having been built up during the low period of flow in the winter. In the past, lack of free-board or slope (or both) has restricted free handling of the water, requiring that increases in the Main Canal discharge be made in small quantities to give the bed sands opportunity to move out and thus increase the water area of the canal. Similar bed-sand phenomena on the Colorado River are generally known; but the necessity is not generally known for the Imperial System to carry large waste-water reserves to cover large fluctuations, the sources of which are on the main canals themselves.

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DISCUSSIONS

MODEL LAW FOR MOTION OF SALT WATER THROUGH FRESH

Discussion

By Messrs. Herbert D. Vogel, and C. E. Grunsky

Herbert D. Vogel,[8] Assoc. M. Am. Soc. C. E. (by letter).[8a]—The ideas presented by Professor O'Brien and Mr. Cherno are particularly refreshing at this time in that they serve so admirably to reflect the broader attitude toward design and the use of hydraulic models, that distinguishes the research work of American from European investigators. The writer has frequently urged abandoning the rigid limits of geometric similitude when dealing with models of rivers and other open channels, for the reason that similarity of effect is of infinitely greater importance than a mere similarity of shape. In practice, every model is distorted in some respects, and the investigator must be on guard continually to avoid overlooking the less obvious distortions of properties and forces while attempting to avoid geometric distortion, which in the last analysis can be held responsible for few important errors.

As the authors suggest, the first and most important step in designing the model is to choose a vertical scale that will result in the production of turbulent flow and properly measurable depths. Following this, consideration must be taken of the size of the prototype, the space available for the model, and the water supply at hand. From a practical standpoint these considerations must determine the horizontal scale, which will approximately equal the ratio of the length of available laboratory floor space to the length of the prototype. The distortion will be the ratio between the vertical and horizontal scales, and other scale values must then be determined to conform dynamically with the originally chosen ratios, keeping in mind continually the main purpose of the investigation.

In the case of experiments relative to the motion of salt water through fresh, the O'Brien–Cherno law is provided as a handy tool to determine the

Note.—This paper by Morrough P. O'Brien and John Cherno, Assoc. Members, Am. Soc. C. E., was published in December, 1932, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: March 1933, by W. E. Howland, Assoc. M. Am. Soc. C. E.

[8] 1st Lieut., Corps of Engrs., U. S. A.; Director, U. S. Waterways Experiment Station, Vicksburg, Miss.

[8a] Received by the Secretary January 18, 1933.

salinity ratio necessary for dynamic similarity when horizontal and vertical scales have been established. The law, of course, could be applied conversely to determine the horizontal scale of the model, having already decided upon the vertical scale and salinity ratio, but it is believed that its greatest utility will be in determining a salinity ratio after arbitrarily selecting the linear relationships. An interesting point is raised in conjecturing the practical limits to which the law may be applied, although there appears, offhand, no valid reason for avoiding what might appear to be extreme distortions. In the absence, however, of cogent reasons for resorting to gross geometric distortion and in the interest of conservatism it would seem best to distort no more than is necessary to insure turbulence and measurable depths. Practically every experimenter has laid down limits of distortion that, in his opinion, must not be exceeded, and it is of more than passing interest to note that the imposed limit in each case tends to vary directly with the size of prototype in which the particular experimenter is most interested. Since the writer has to deal largely with flood control and navigation problems relative to the Mississippi River and its tributaries, he is inclined to be rather generous in the matter.

In closing, it is desired to call attention to the notations and symbols used at the United States Waterways Experiment Station, suggesting that their general adoption in the United States might considerably simplify the reading and understanding of technical papers on the subject of similitude. As used by the U. S. Waterways Experiment Station: L_m = a horizontal length in the model; L_n = a corresponding length in Nature (prototype); $l = \frac{L_m}{L_n}$ = the horizontal linear scale; $d = \frac{D_m}{D_n}$ = the vertical linear or depth scale; $v = \frac{V_m}{V_n}$ = the velocity scale; etc. From the foregoing it is seen that capital letters with suitable subscripts represent dimensions in the model and prototype, respectively, while small letters represent their ratios, or the scale values. Continuing: q = discharge scale; a = area scale; e = scale of sand-grain diameters; r = hydraulic radius scale; etc.

The authors are to be congratulated for the able way in which they have handled a difficult subject and for their courage in daring to deviate from well beaten European paths. The writer has found that the same principles are suitable to investigations of navigation works in rivers[9] and has verified to his own satisfaction the feasibility of designing hydraulic models to preserve dynamic similarity under all conditions of operation.

C. E. GRUNSKY,[10] PAST-PRESIDENT, AM. SOC. C. E. (by letter).[10a]—The experiments described in this paper, some of which it was the writer's privilege to have attended, were interesting in giving occular demonstration of the behavior of two vertical surfaces of liquids of different density in contact

[9] *Civil Engineering*, August, 1932, p. 467.

[10] Cons. Engr. (C. E. Grunsky Co.), San Francisco, Calif.

[10a] Received by the Secretary January 28, 1933.

with each other when freed from restraint. However, the conditions at a lock when a gate between a fresh-water and a salt-water compartment is to be opened, were not reproduced at the model, and it is questionable whether the deductions made by the authors should not be subjected to further review. Personally, the writer does not think it worth while.

In the demonstration with the experimental model the water surface on both sides of the partition separating the compartments of different densities were at the same level. There was consequently greater aggregate pressure from top to bottom against the partition on the side of the denser liquid than against the other side. At a lock-gate—the device which is being studied—the opening of the gate does not take place until the total water pressures on the two sides are practically equal. This occurs when the fresh water attains a superelevation over the ocean water of about 1.3 per cent. In a lock with 40 ft of salt water on the sill, the superelevation of the water in the fresh-water compartment would be somewhat more than $\frac{1}{2}$ ft.

At the surface of the salt water there would be a velocity head of about 0.52 ft impelling the fresh water against the salt water. At 20 ft. above the gate-sill, the velocity head, expressed in fresh water, would be the same on the salt-water side as on the fresh-water side, and at the sill the velocity head, expressed in fresh water, would be 0.52 ft greater on the salt-water side.

Sudden removal of the partition, of course, would cause an over-run of fresh water in one direction, while there is an under-flow of salt water in the other direction, although necessarily somewhat checked in its velocity of flow by the superelevation of the fresh water, a factor which has not been taken into consideration by the authors.

It is an interesting fact that under certain physical conditions, such superelevation as here referred to may be fairly permanent in locations where there is a gradual transition from fresh water to ocean water. This fact was brought to the attention of the Federal and State engineers who were in charge of studies for a San Francisco Bay salt-water barrier, by the writer in 1931.

Take, for example, any long narrow tidal estuary in which there is a fair depth and considerable tidal movement. Here, as long as there is any material accession of fresh water, there will be a gradual approach, throughout a long stretch of estuary from fresh-water density to ocean density. If, now, the great block of brackish water in such an estuary is regarded as a block of foreign material interposed between the fresh and the salt water—a gate, in other words, that may be 40 or 50 miles thick up and down stream—then it will at once be apparent that there can be no equilibrium unless the fresh water at the upper end of this block is at a higher level than the salt water at its down-stream end. The brackish water, in other words, has a gradient from its fresher to its saltier end. Despite this gradient, the area of brackish water will be in substantial equilibrium.

Take, for example, the Upper San Francisco Bay region and the delta of the Sacramento and San Joaquin Rivers. At the fall low-water stage of these rivers, brackish water fills Suisun Bay which lies at, or just below, their junction. Brackish water is pushed by the tide back and forth in the San

Joaquin River for about 20 or 30 miles in a channel which for a long distance has depths exceeding 50 ft. In this long river stretch, and on down through Suisun Bay and the Straits of Carquinez, there is a gradual increase of water density from fresh water (specific gravity 1.00) to ocean water (specific gravity 1.026).

In the Straits of Carquinez below Suisun Bay the water has generally a maximum depth of from 50 to 60 ft. The tidal range here averages, at low stages, 5 to 6 ft, and tidal currents are strong. There is no pocketed salt water. Transition from the fresh to the salt is gradual throughout the stretch of water here under consideration. It follows, therefore, that in the lower delta of these rivers and through Suisun Bay to wherever full-strength ocean water penetrates, there will be an inclined stable water surface with a superelevation at up-stream points of about ($\frac{1}{2} \times 60 \times 0.026 =$) 0.8 ft.

Sea level, for example, approximated by mean tide at Stockton, Calif., above the reach of brackish water, but on the waters of the Lower San Joaquin River, will be about 0.8 ft higher than true sea level because of the depth (60 ft, or thereabouts) throughout which the lighter fresh water must offset the greater pressure of the salt water in the down-stream delta channels.

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DISCUSSIONS

WIND STRESS ANALYSIS SIMPLIFIED

Discussion

BY MESSRS. FRANCIS P. WITMER, ELMER K. TIMBY, N. A. RICHARDS, JOHN B. LETHERBURY, FREDERICK MARTIN WEISS, AND RAYMOND C. REESE

FRANCIS P. WITMER,[8] M. AM. SOC. C. E. (by letter).[8a]—Apparently, by making a reasonably close preliminary estimate of the horizontally deflected positions of the various joints, it is expected that the effect of "side-sway" may be found more expeditiously by Professor Grinter's method than by the process of repeatedly equalizing shears in each story, as described in the Second Progress Report of Sub-Committee No. 31, Committee on Steel, of the Structural Division, on Wind-Bracing in Steel Buildings.[9]

Apparently, the effect of vertical translation of joints, due to column distortions under direct wind stress, may be found either at the same time with the effect of horizontal displacements by making this consideration a part of the original analysis, or it may be included in a second analysis covering only these vertical translations.

If the analysis of the frame is first made by the method described in the progress report previously mentioned, neglecting column axial distortions, the horizontal movements will be fully cared for. A second analysis by the Cross method may then be performed for the vertical distortions only, considering these as producing secondary moments, and following Professor Grinter's method for obtaining the starting fixed-end moments in cross-girders. Should this second analysis result in new column axial distortions of sufficient size to justify it, a third analysis, and even a fourth, may be necessary to determine with sufficient accuracy the moments resulting from vertical displacements. The combination of these secondary moments (and, if required, those also of higher orders) with moments from the original analysis will produce the final moments in the frame.

NOTE.—The paper by L. E. Grinter, Assoc. M. Am. Soc. C. E., was presented at the meeting of the Structural Division, New York, N. Y., January 19, 1933, and published in January, 1933, *Proceedings*. This discussion is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

[8] Director, Dept. of Civ. Eng., Univ. of Pennsylvania, Philadelphia, Pa.

[8a] Received by the Secretary January 18, 1933.

[9] *Proceedings*, Am. Soc. C. E., February, 1932, p. 226, Step 4.

Professor Grinter describes a solution of secondary stresses in a truss based upon his method, using a Williot diagram to obtain the translations of joints from which starting fixed-end moments are found. The writer described[10] a similar method in 1932.

ELMER K. TIMBY,[11] JUN. AM. SOC. C. E. (by letter).[11a]—In this paper Professor Grinter treats wind stresses in building frames of both regular and irregular proportions. To develop the methods presented, certain assumptions were made, and the purpose of this discussion is to inquire as to whether variations from these primary assumptions materially affect the reliability of the results obtained when using the simplified method.

The first assumption made in the treatment of regular symmetrical frames is that the deflections in all stories are in proportion to the story heights, or, mathematically, that $\frac{\Delta}{h}$ is a constant.

In the development of the method for estimating deflections of irregular building frames Professor Grinter assumes that the total moment in any column other than the basement column is equally divided between the two ends. This is equivalent to assuming a point of contraflexure at mid-story height. For the basement column, the end moments are assumed to be 40% of the total at the top and 60% of the total at the bottom, which slightly raises the point of contraflexure.

It is stated in the paper that a wind bent of ideal proportions should have identical columns in each story, and stories of constant height; and, further, that the K-values of the columns and girders should increase from the top downward in direct proportion to the story shears.

In 1930, Messrs. H. W. Coultas and V. H. Lawton, using a Beggs deformeter apparatus, conducted a most complete experimental analysis of a building frame.[12] The writer has calculated by the area-moment method the deflections of this structure, using the moments determined by the experimental analysis.

It will be seen from Fig. 9(a) that the frame is six stories high, totaling 79 ft to the roof beams, and that its width of 51 ft is equally divided into two bays, making it symmetrical about the center column, $C(2)$. The wind load was assumed to have an intensity of W lb per unit of area above the point, A, shown slightly above the center of the left column, and three-fourths of W lb below Point A. The shaded areas on Fig. 9(a) represent the moments of inertia of the members. It will be observed that the values for the columns increase from top to bottom quite uniformly; that the story heights are nearly equal, having a maximum variation of 1 ft; and, further, that the frame is regular and symmetrical. Therefore, this frame selected at random satisfies to a remarkable degree the conditions stipulated by Professor Grinter as defining an ideal frame.

[10] *Engineering News-Record*, January 28, 1932.

[11] Instr., Eng., School of Eng., Princeton Univ., Princeton, N. J.

[11a] Received by the Secretary January 20, 1933.

[12] *The Structural Engineer* (London), October, 1930.

Fig. 9(*b*) is a shaded moment diagram for the center column, $C(2)$. It will be observed that the columns in the top three stories have no point of zero moment and, therefore, no point of contraflexure, and that the second-

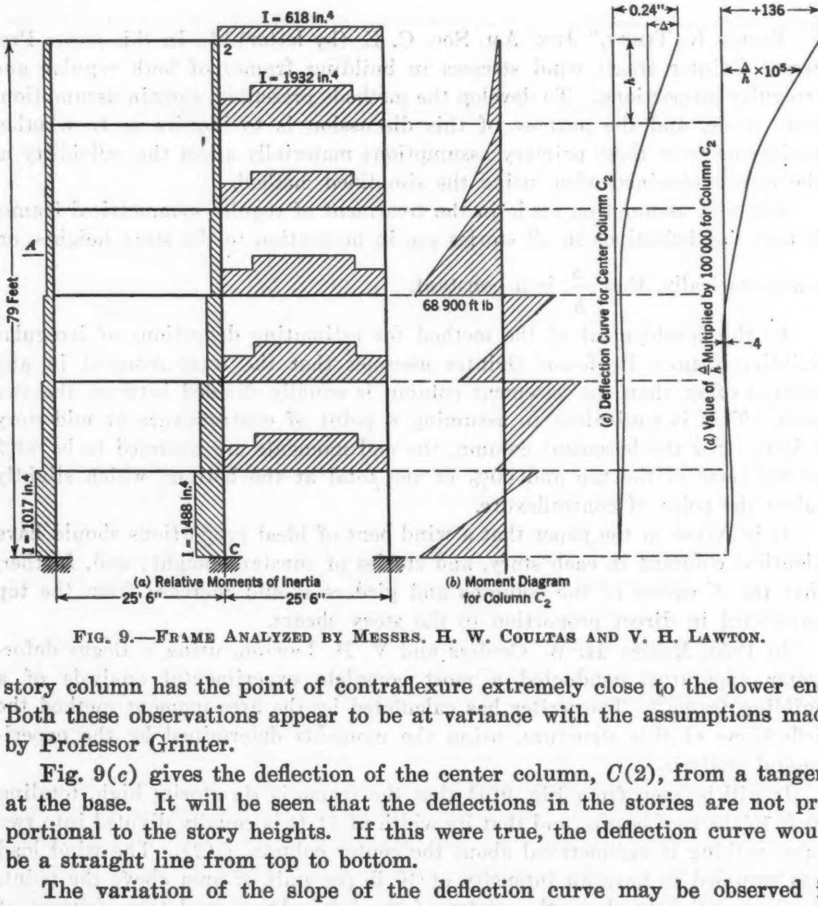

FIG. 9.—FRAME ANALYZED BY MESSRS. H. W. COULTAS AND V. H. LAWTON.

story column has the point of contraflexure extremely close to the lower end. Both these observations appear to be at variance with the assumptions made by Professor Grinter.

Fig. 9(*c*) gives the deflection of the center column, $C(2)$, from a tangent at the base. It will be seen that the deflections in the stories are not proportional to the story heights. If this were true, the deflection curve would be a straight line from top to bottom.

The variation of the slope of the deflection curve may be observed in Fig. 9(*d*). These values of $\frac{\Delta}{h}$, which are assumed to be constant in Professor Grinter's paper, change from $+136$ to -4. These values were computed, assuming the tangent at the base to be vertical.

Any method which reduces the amount of time or labor required for the solution of a problem, and which, at the same time, yields a high degree of accuracy, is to be highly commended. The writer would like to inquire, however, whether or not these apparently radical differences between assumptions and conditions materially affect the accuracy obtainable with the author's simplified method.

The writer felt, before making this investigation, that the assumption, $\frac{\Delta}{h}$ = a constant, might not be reliable for frames of great height, because of the important effect of column strains. He was surprised, however, to find that the assumption does not appear to be valid for some simple regular frames of low height. A discussion of this particular point by Professor Grinter would be appreciated.

N. A. Richards,[13] M. Am. Soc. C. E. (by letter).[13a]—In this paper Professor Grinter offers a method for the analysis of continuous frames in which joint displacements occur. After illustrating the applicability of this method to other frames, he shows how it may be used in the design of high building frames subject to wind. Like other methods of analysis, it presupposes the existence of a design to analyze, and this design must be prepared by some method wholly distinct from this method of analysis.

In buildings of moderate height and dimensions, in which the bracing is not the prime feature of the design, it might often happen that this method would prove useful in a check to determine whether the ordinary members in the building frame would be overstressed from an assumed wind. However, if the building frame presents a major wind problem, due to its height or narrowness, it seems to the writer that it is approaching the design in a backhanded way to create it by some approximate method, and then analyze and revise it by a method such as is suggested.

In the design of structural members generally, such as girders, beams, and columns, subject to dead and live loads, the designer proceeds in a direct manner to the selection of members sufficient for the conditions of loading, bending, and shear, and without the necessity of a review of some other method to determine whether the members will act as designed.

In their Third Progress Report the members of Sub-Committee No. 31, Committee on Steel, of the Structural Division, on Wind-Bracing in Steel Buildings indicate[14] their belief that they have suggested a method of approach to wind-bracing design, which enables the designer to proceed in as direct a manner to the selection of the members of such a frame, and with no more necessity for review by another analytical method.

It is not the purpose of this comment to criticize the merits of Professor Grinter's method, but rather to bring out the fact that it is of value principally as a method of analysis, and not as a tool of design.

John B. Letherbury,[15] Jun. Am. Soc. C. E. (by letter).[15a]—The author's method of determining wind moments and deflections supplies a rapid means of solving an otherwise laborious problem. The writer was particularly interested in the conception of a combined girder and column replacing the wind

[13] Vice-Pres., Purdy & Henderson Co., New York, N. Y.

[13a] Received by the Secretary January 28, 1933.

[14] See abstract of report in *Civil Engineering*, March, 1933.

[15] Philadelphia, Pa.

[15a] Received by the Secretary February 1, 1933.

bent, and he desired to experiment with the method on a frame with which he is familiar.

The frame shown in Fig. 10(*a*) was chosen originally for use in comparing a statically determinate method with the method of moment and shear adjustment suggested[16] by Sub-Committee No. 31, Committee on Steel, of

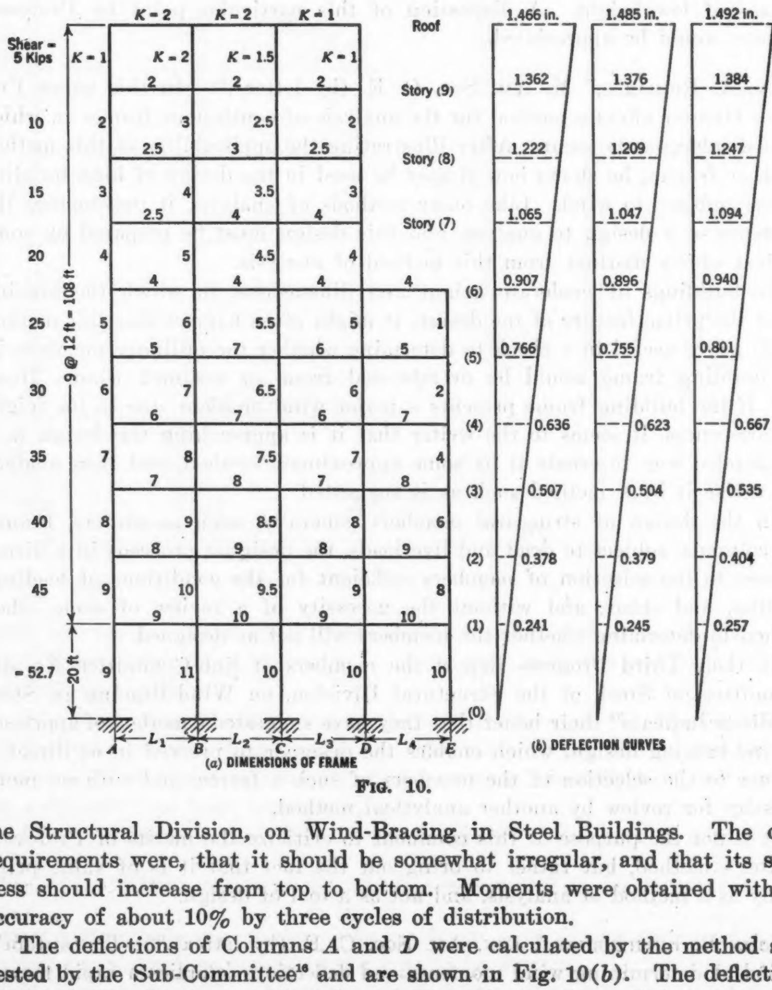

FIG. 10.

the Structural Division, on Wind-Bracing in Steel Buildings. The only requirements were, that it should be somewhat irregular, and that its stiffness should increase from top to bottom. Moments were obtained with an accuracy of about 10% by three cycles of distribution.

The deflections of Columns *A* and *D* were calculated by the method suggested by the Sub-Committee[16] and are shown in Fig. 10(*b*). The deflections of the combined column and girder calculated by Professor Grinter's method are also shown. All three of the curves approximate a straight line. The combined column and girder deflects but little more than either of the columns, the maximum difference being about 7 per cent. This would seem to

[16] Second Progress Report of Sub-Committee No. 31, Committee on Steel, of the Structural Division, on Wind-Bracing in Steel Buildings, *Proceedings*, Am. Soc. C. E., February, 1932, p. 213.

indicate that the fixed-end moments could be considered as proportional to the stiffness of the columns.

A slight break occurs in the deflection curves at the top of the fifth story, indicating that the effect of the set-back occurs at that point, An increase in uniform wind loading on the upper stories would cause a similar break in the curve which would then approximate two straight lines joined by a curved line. A much higher frame might show a deflection curve more

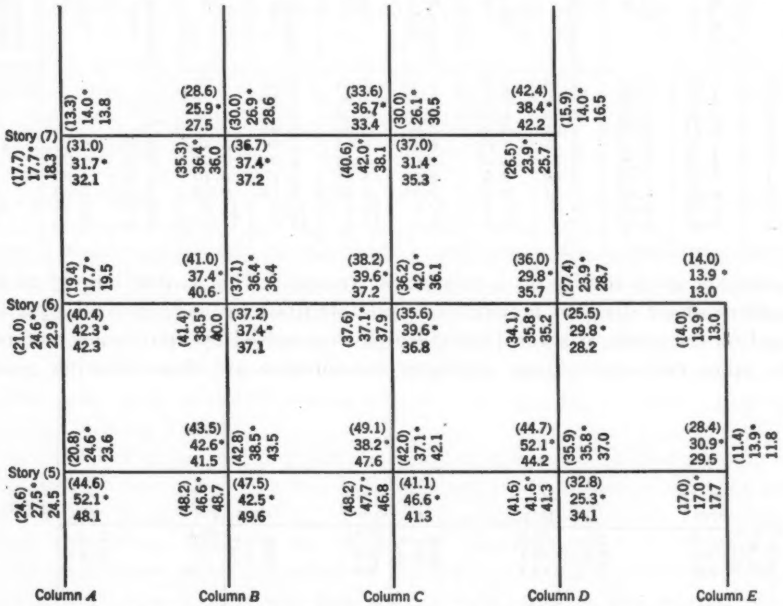

FIG. 11.—MOMENTS IN SET-BACK PORTION COMPARED BY THREE METHODS.

nearly approximating a flat parabola than a straight line, but as Professor Grinter has pointed out, the story deflections would be sufficiently uniform to warrant the assumption of fixed-end moments proportional to the K-values of the columns.

In Fig. 11, the moments, in foot-kips, are shown for the set-back portion of the frame. The moment in each case faces the member to which it is applied. Values in parenthesis—thus, (10.00)—are by the Grinter method; those marked by an asterisk—thus, 10.00*—are by the Bowman method; and the remainder are by three cycles of moment and shear distribution. Fixed-end moments were calculated from the story deflections and carried through two cycles of distribution, which brought the final moments into close agreement with those obtained by three cycles of moment and shear distribution. The criterion ratio for the sixth story was 1.05.

The statically determinate method[17] used is shown for comparison. It was devised by H. L. Bowman, M. Am. Soc. C. E. It has been found to give

[17] "Structural Theory," by Hale Sutherland and Harry Lake Bowman, Members, Am. Soc. C. E.

TABLE 4.—COMPARISON OF COLUMN SHEAR, IN KIPS, CALCULATED BY THE STATICALLY DETERMINATE AND THE SIMPLIFIED METHODS

Story	COLUMN A		COLUMN B		COLUMN C		COLUMN D		COLUMN E		Maximum difference in percentage of simplified method
	Statically determinate method	Simplified method	Statically determinate method	Simplified method	Statically determinate method	Simplified method	Statically determinate method	Simplified method	Statically determinate method	Simplified method	
10	0.98	0.83	1.96	1.91	1.46	1.48	0.61	0.77	.....	.....	21
9	1.78	1.87	3.29	3.32	3.16	3.09	1.78	1.77	.....	.....	5
8	2.59	2.51	4.98	4.85	4.84	·5.06	2.59	2.58	.....	.....	4
7	2.94	3.19	6.07	6.03	7.00	6.40	3.99	4.32	.....	.....	9
6	4.10	3.48	6.41	7.02	6.19	6.65	5.97	5.83	2.32	2.12	15
5	4.59	4.22	7.77	7.85	7.95	7.85	6.94	7.07	2.84	2.79	9
4	5.34	5.00	8.60	8.60	8.40	8.45	8.17	8.15	4.50	4.80	7
3	6.04	5.60	9.64	9.80	9.44	9.50	9.24	9.70	5.63	5.60	8
2	6.73	6.00	10.7	11.0	10.5	10.7	10.3	11.0	6.74	6.14	12
1	8.61	8.30	12.1	12.2	11.3	11.1	11.3	11.4	9.46	9.50	4

reasonably good results on a number of frames. This is due largely to the excellent shear division between columns obtained by an empirical equation based on the study of several bents which were solved by more exact methods. The other two assumptions necessary for solution are those locating points

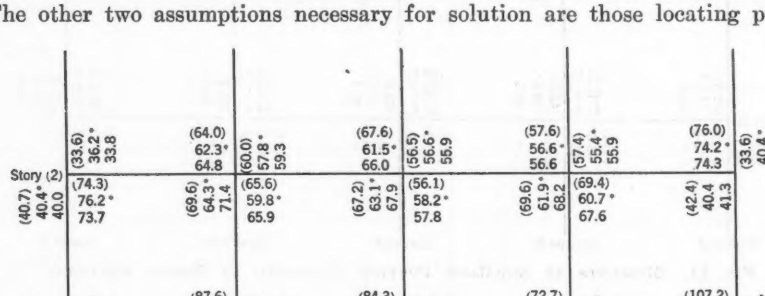

FIG. 12.—COMPARISON OF MOMENTS IN THE TWO LOWER STORIES.

of contraflexure in the columns and girders. The writer mentions this method because of its good results and its flexibility in application to design. Table 4 gives a comparison of the column shears, in kips, in the frame shown.

In Fig. 12, the comparison for the two lower stories is given, the units and symbols being the same as those in Fig. 11. In this, the fixed-end

moments calculated by the simplified method were carried through four cycles to a criterion ratio of 0.96. The statically determinate method is within 20% of agreement.

The writer believes that the simplified method reduces the labor of wind-stress analysis and the calculations of frame deflections to a minimum consistent with reasonable accuracy. The conception of a combined column and girder may be developed into a yardstick for the measurement of tower deflection and vibration. The natural period of the frame could be calculated from its mass and the deflections of the combined column and girder, assuming harmonic motion and some possible gust forces. The amplitudes and frequencies thus obtained would be in excess of the actual ones in the finished building. It seems logical to attack the problem by means of Professor Grinter's conception.

FREDERICK MARTIN WEISS,[18] JUN. AM. SOC. C. E. (by letter).[18a]—The author is to be commended for his efforts to simplify a solution to one of the most complex problems arising in structural engineering. Although the system of analysis presented may deserve considerable space in college textbooks and discussion in academic circles, it is scarcely likely to prove to be the "short-cut" long awaited by practising structural engineers. Conclusions as to its practical worth must not be drawn from the sample calculations relating to the lower parts of the Wilson and Maney bent and the American Insurance Union Building bent.

In the first place, nothing but the simplest method of wind-stress analysis can possibly be applied to a tall building comprising a vast number of members requiring such attention. The three sample calculations presented by Professor Grinter, contain 14, 43, and 21 members, respectively, and from these an estimate may be obtained of the time and labor required per member. Then, when one is informed that such a skyscraper as the 56-story Irving Trust Company Building, in New York City, contains approximately 6 200 members in the wind-resisting part of the frame, it becomes evident that, because of the limited time allowed a structural engineer for the complete design of a steel-framed building, "exact" or "semi-exact" analyses must be abandoned and resort made to less tedious and laborious methods. This is not a plea for the adoption of quick and inaccurate wind-stress calculations, but merely a revelation of the magnitude of the task confronting designers of high buildings.

In the second place, Professor Grinter seems to have overlooked one of the peculiar characteristics of modern tall buildings. Unfortunately, these characteristics do not occur in the Wilson and Maney bent, or in the American Insurance Union Building bent, which were necessarily chosen because slope-deflection analyses were available for them. In general, it will be found that in tall buildings the stiffness ratios of the lower-story columns will be many times the stiffness ratios of the girders connected to them. In applying the simplified method of successive corrections to the analyses of such building

[18] Passaic, N. J.

[18a] Received by the Secretary February 17, 1933.

bents, the convergence of approximations will be very slow because, upon balancing any joint, very little of the unbalanced moment will be distributed to the girders. Since the original fixed-end moments of the girders are zero, and since the balancing moments are small and the carry-over moments are even smaller, it is evident that a great many approximations must be performed before the final balanced moments of the girders at any joint are sufficiently large to equal the sum of the column moments at the same joint.

TABLE 5.—DIMENSIONS AND CHARACTERISTICS OF THE IRVING TRUST COMPANY BUILDING BELOW THE NINTH FLOOR

Story	Height, in feet	COLUMNS						GIRDERS				
		MOMENTS OF INERTIA, IN INCHES[4]										
		A	*B*	*C*	*D*	*E*	*F*	*A-B* (span, 26.08 ft)	*B-C* (span, 15.83 ft)	*C-D* (span, 27.00 ft)	*D-E* (span, 15.83 ft)	*E-F*; (span, 19.58 ft)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)
	12.5	5 609	15 969	15 314	15 314	13 317	4 476					
8								2 735	2 184	2 234	2 184	2 457
	12.5	6 421	16 717	16 717	16 717	14 600	5 221					
7								2 735	2 184	2 234	2 184	2 457
	12.5	6 421	16 717	16 717	16 717	14 600	5 221					
6								2 735	2 457	2 234	2 184	2 457
	12.5	8 238	17 501	17 501	17 501	15 314	6 011					
5								3 021	2 457	2 234	2 457	3 021
	12.5	8 238	17 501	17 501	17 501	15 314	6 011					
4								4 380	2 457	2 234	2 457	3 344
	12.5	9 785	18 518	18 518	18 518	15 969	8 238					
3								4 721	3 021	4 721	3 021	3 670
	16.21	9 785	18 518	18 518	18 518	15 969	8 238					
2								6 050	3 217	6 050	3 217	4 985
	14.00	13 621	19 894	20 374	19 894	17 379	9 785					
1								6 050	3 217	6 050	3 217	4 985
	18.00	13 621	19 894	20 374	19 894	17 379	9 785					
*G**								4 045	1 543	9 306	1 543	1 954
	14.00	†	21 814	22 774	22 774	18 518	†					
*B**								6 664	2 457	8 301	2 184	2 457
	12.00	†	21 814	22 774	22 774	18 518	†					
*SB**								1 954	1 954	3 596	1 954	1 954
	12.33	†	22 774	23 318	23 318	18 998	†					

* *G* = ground floor; *B* = basement: and *SB* = Sub-basement. † Foundation walls.

A rough estimate of the rapidity with which convergence of the approximations may be expected can be obtained from the quotient of the sum of the stiffness ratios of the columns intersecting at a joint divided by the sum of the stiffness ratios of the girders intersecting at the same joint. The larger these quotients are, the slower will be the rate of convergence. In Fig. 13 values of these factors are plotted for the lower eight floors of the Wilson and Maney bent and for the lower eight floors of the American Insurance Union Building bent, together with values for the lower eleven floors of a bent selected through the tower of the Irving Trust Company Building. The necessary dimensions, in feet, and moments of inertia, in inches[4], for the Irving Trust Company Building are arranged for reference in Table 5. It will be noticed in Fig. 13 that the values for the Wilson and Maney bent and for the American Insurance Union Building bent are very much smaller, in general, than the values for the Irving Trust Company Building.

In conclusion, therefore, it may be stated that to attain a desired degree of accuracy, the practical application of this variation of the Cross method

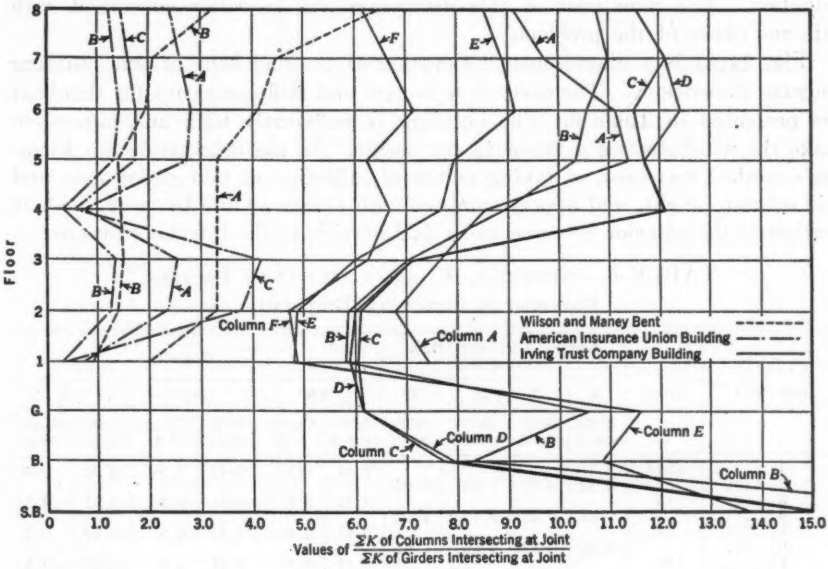

FIG. 13

may become such a Herculean undertaking as to overshadow completely the task performed by Professors Wilson and Maney in solving sixty simultaneous equations.[19]

RAYMOND C. REESE,[20] ASSOC. M. AM. SOC. C. E. (by letter).[20a]—In applying Professor Cross' method of moment distribution and Professor Grinter's simplification, the designer is mainly interested in how quickly he can obtain reasonably accurate results. From the nature of the problem, no "exact" solution can be expected; nor is it necessary.

The work required is dependent upon the speed with which the moment distribution converges; while the accuracy can only be determined by agreement with other recognized methods of attack. The speed of convergence, again, depends upon the relative stiffness of the members, particularly the ratio of story-column stiffness to the stiffness of one tier of beams.

Most of the recorded examples are fairly regular frames, as that is the only kind amenable to the so-called "exact" methods that are used as a check. In actual work, however, it is seldom possible to make such simple and regular frames. This is particularly true of reinforced concrete buildings, where the designer has an unlimited choice of sections.

The writer was making a review of the wind stresses in an ordinary fifteen-story reinforced concrete hotel building for another purpose, and it seemed an excellent chance to determine the speed of convergence of the dis-

[19] *Bulletin No. 80*, Eng. Experiment Station, Univ. of Illinois, Urbana, Ill.

[20] Secy. and Chf. Engr., The Hausman Steel Co., Toledo, Ohio.

[20a] Received by the Secretary March 3, 1933.

tribution. The remainder of this discussion will be solely concerned with this one phase of the problem.

Fig. 14(*a*) is a diagrammatic elevation of the end bent of this building showing dimensions. The carry-over factors and stiffness ratios for this bent are presented in Table 6. The building is sufficiently high and narrow to make the wind stresses a factor in the design. In the original design Fleming's method was used, of taking points of inflection at mid-girder span and mid-column height, and apportioning column shears on the basis of one-half portion to the exterior columns and a full portion to the interior columns.

TABLE 6.—STIFFNESS, *K*, AND CARRY-OVER FACTORS, REINFORCED CONCRETE BUILDING

Item No.	Floor	Columns (Carry-Over Factor, 0.5)				Girders					
		A	*B*	*C*	*D*	*AB*		*BC*		*CD*	
		Stiffness, *K*	Stiffness, *K*	Stiffness, *K*	Stiffness, *K*	Stiffness, *K*	Carry-over	Stiffness, *K*	Carry-over	Stiffness, *K*	Carry-over
1	Roof					0.41	0.5	0.41	0.5	0.97	0.5
2		0.32	0.32	0.95	0.95						
3	15					0.41	0.5	0.41	0.5	0.97	0.5
4		0.32	0.32	2.10	1.52						
5	14					0.41	0.5	0.41	0.5	0.97	0.5
6		0.32	0 83	4.60	1.61						
7	13					0.41	0.5	0.41	0.5	0.97	0.5
8		0.56	1 52	7.80	2.32						
9	12					0.65	0.45	0.65	0.45	2.00	0.5
10		0.95	2.32	9.00	2.56						
11	11					0.65	0.45	0.65	0.45	2.00	0.5
12		1.52	3.61	9.00	3.61						
13	10					0.65	0.45	0.65	0.45	2.00	0.5
14		2.10	4.43	9.00	3.61						
15	9					0.65	0.45	0.65	0.45	2.00	0.5
16		2.32	5.14	9.00	3.61						
17	8					0.65	0.45	0.65	0.45	2.00	0.5
18		3.61	5.14	9.00	3.61						
19	7					1.00	0.667	1.00	0.667	2.00	0.5
20		4.71	5.14	26.60	3.61						
21	6					1.00	0.667	1.00	0.667	2.00	0.5
22		5.14	5.14	26.60	4.32						
23	5					1.00	0.667	1.00	0.667	2.00	0.5
24		5.14	7.06	26.60	4.32						
25	4					1.00	0.667	1.00	0.667	2.00	0.5
26		5.14	7.06	26.60	7.06						
27	3					1.00	0.667	1.00	0.667	2.00	0.5
28		5.14	7.06	26.60	7.06						
29	2										

The size and shape of many of the members were determined by architectural considerations. The column adjoining the stairs was of long, narrow cross-section, to clear the stair shaft, and was built with three spirals. The depth of spandrels was fixed by the legal height of the window heads. They could not be inverted because of the steam return lines buried in the walls; but were deepened at the ends to afford as much wind resistance as possible. Double reinforcement was used in these beams.

This frame, therefore, is typical of what can be expected in reinforced concrete buildings designed for maximum economy, and meeting the necessary requirements as to space and clearances. The members differ widely in their relative stiffness. In the lower stories the columns are much stiffer than the girders. All these factors operate to slow down the convergence of the moment distribution.

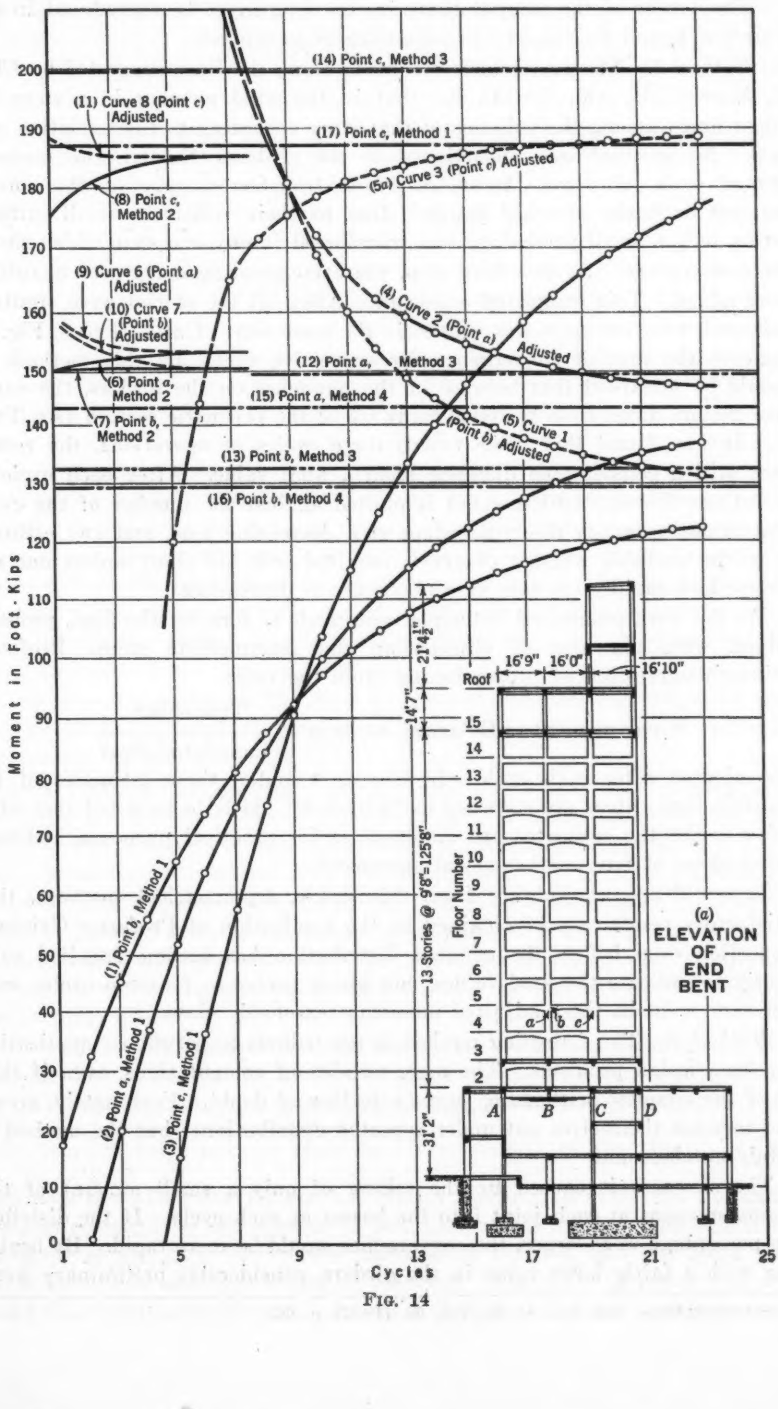
(14) Point c, Method 3
(11) Curve 8 (Point c) Adjusted
(17) Point c, Method 1
(5a) Curve 3 (Point c) Adjusted
(8) Point c, Method 2
(9) Curve 6 (Point a) Adjusted
(10) Curve 7 (Point b) Adjusted
(4) Curve 2 (Point a) Adjusted
(12) Point a, Method 3
(6) Point a, Method 2
(15) Point a, Method 4
(7) Point b, Method 2
(5) Curve 1 (Point b) Adjusted
(13) Point b, Method 3
(16) Point b, Method 4
(1) Point b, Method 1
(2) Point a, Method 1
(3) Point c, Method 1
Moment in Foot-Kips
Cycles
200
190
180
170
160
150
140
130
120
110
100
90
80
70
60
50
40
30
20
10
0
1
5
9
13
17
21
25
16'9"
16'0"
16'10"
Roof
21'4½"
14'7"
13 Stories @ 9'8"=125'8"
31'2"
Floor Number
15
14
13
12
11
10
9
8
7
6
5
4
3
2
a
b
c
A
B
C
D
(a)
ELEVATION OF END BENT

FIG. 14

The details of the computations are too lengthy to be reproduced in full. Points *a, b,* and *c* (Fig. 14(*a*)) were selected as typical.

Method 1.—The first attack was made along the lines suggested by Clyde T. Morris,[21] M. Am. Soc. C. E.; that is, the wind moment in a story was apportioned among the columns of that story according to their relative stiffness. No moment was apportioned to the girders. Second, the moments around each joint were balanced, which transferred some of the column moment into the attached beams. Due to their relatively small stiffness ratios, only a small percentage was transferred in any one step, which slowed the convergence. As the third step, the corresponding carry-over quantities were added. This completed one cycle. After all the stories were similarly balanced, a second cycle was started in the same way. Curves 1 to 3, Fig. 14, indicate the rate of convergence for successive cycles by this method. It should be remarked that because of the haunches on the girders, the carry-over factors differ from the customany 0.5 of the prismatic section (see Table 6). It was found that after twenty-three cycles of operations, the results were still a considerable distance from a final value. After each cycle of operations the resultant moment is plotted against the number of the cycle. The values approach the true values at a decreasing rate, and any estimate as to the probable number of cycles required will fall short unless one can foresee how rapidly the rate of convergence is decreasing.

As the work progressed, attempts were made to forecast the final, probable values, with the idea of eliminating the intermediate steps. Professor Grinter's suggestion of proportioning from the rule:

$$\text{Final moment} = \text{Balanced moment} \times \frac{\text{True shear}}{\text{Calculated shear}}$$

was adopted after each cycle. In Curves 4 and 5 these estimates of the probable final values are recorded as "adjusted." It is to be noted that after a few cycles the estimates are amply close for practical purposes, although rather short of true mathematical agreement.

It would appear, judging from this single, asymmetrical instance, that satisfactory results can be obtained by the application of Professor Grinter's proportion even before the moment distribution has become steadied sufficiently to indicate the final values; but about twelve to fourteen cycles were necessary to make these adjusted moments reasonably close.

Method 2.—The foregoing method is too tedious to permit its application to actual design problems. The mere number of computations, even if they are of the simplest arithmetic, leaves a feeling of doubt. Fortunately, errors tend to work themselves out under repeated distributions; but the method is unduly cumbersome.

The slowness is caused by the taking of only a small amount of the column moment at each joint into the beams at each cycle. If the distribution percentage were larger, the convergence would be more rapid. By beginning with a fairly large value in the girders, considerable preliminary work

[21] *Transactions*, Am. Soc. C. E., Vol. 96 (1932) p. 66.

would be eliminated. Arbitrary values could be assumed, but the results might then tend to converge on these values without their being the true results.

It was decided to start with the values obtained by the ordinary approximate solution. These values were first adjusted because it was impossible to have the same moments in the haunched beams and in the full depth spandrel. Consequently, about 10% was deducted from the approximate moments in each of the haunched spandrels and the corresponding amount was added into the full depth spandrel. This unbalanced the joints, and a rough adjustment of the column moments was made to keep each joint approximately in balance. In some cases, this was impossible. From a common sense viewpoint it was unreasonable to have a greater moment at the top of a column than at the bottom, and yet no other solution would balance the joint. In such cases values were adopted that were as nearly in balance as possible, and, at the same time, as nearly as possible consistent with each other. No careful attempt was made to fill either criterion, as it was felt that a few cycles would adjust the discrepancies.

With these trial values as a start, the same cycle of operations as in Method 1 was gone through seven times. With the girder moments assumed at something like their true values, the convergence is much more rapid. In Curves 6, 7, and 8, are plotted the complete set of computations through seven cycles for the same points, *a, b,* and *c.* Here, again, the reproduction of all the detail figures is of no help in visualizing the results. These curves offer a comparison with the previous method.

The approximations of the true final values by the proportion of true shear to calculated shear was applied to the moments in this case, and plotted as Curves 9, 10, and 11, in Fig. 14.

As the values were assumed approximately in balance around each joint and with the columns taking approximately the story moment, the convergence was quick—but not, apparently, on the best, true values. There was a marked tendency to close on the original assumptions. While the adjusted results appear fairly good, the method is mathematically unsound, and the apparent value is based only on a rather fortunate guess in setting up the original assumptions. Actually, the angles between beams and columns are not maintained as true right angles in this method; some relative angle change takes place.

***Method 3.*—This method followed exactly the author's suggestions for very irregular bents; that is, the *r* and *n* values for each floor were computed and the total story moment was distributed as suggested. The *K* values in Table 6 are relative only, and are about one-forty-seventh of the true $\frac{I}{L}$ values. This does not affect the figures, but simply reduces all values in this ratio. For this reason, it is not possible to plot the various cycles. The results for the three points, *a, b,* and *c,* are recorded in Table 7, and the speed with which the figures converge is readily apparent. Six cycles of operations completed the distribution at each point to a precision of about 1 to 2 per cent.**

The final results at Points *a, b,* and *c* are computed by multiplying the values obtained as described, by the ratio of the true shear to the calculated shear; that is, by 7.37 for the fourth-story columns, 8.50 for the fifth-story columns, and the average for the fifth-floor beams.

TABLE 7.—VALUES OF DISTRIBUTED MOMENT AT POINTS *a, b,* AND *c* IN FIG. 14(*a*)

Cycle	*A* Distributed	*A* Result	*B* Distributed	*B* Result	*C* Distributed	*C* Result
	......	+116.0	......	0	......	+436.0
	−78.5	......	−12.2	......	−310.0	......
1.......	−39.3	− 1.8	− 7.4	−19.6	−155.0	− 29.0
	+16.8	......	+ 2.4	......	+ 37.3	......
2.......	+ 8.4	+ 23.4	+ 0.8	−16.4	+ 18.7	+ 27.0
	− 2.0	......	− 0.3	......	+ 2.5	......
3.......	− 1.0	+ 20.4	+ 0.1	−16.6	+ 1.3	+ 30.8
	− 0.1	......	0.0	......	− 3.3	......
4.......	− 0.1	+ 20.2	− 0.1	−16.7	− 1.7	+ 25.8
	+ 0.2	......	0.0	......	+ 1.2	......
5.......	+ 0.1	+ 20.5	0.0	−16.7	+ 0.6	+ 27.6
	− 0.1	......	0.0	......	− 0.3	......
6.......	− 0.05	+ 20.35	0.0	−16.7	− 0.2	+ 27.1

The resultant moments were computed, as follows:

$$M_a = -20.35 \times 7.37 = 149.9$$

$$M_b = -16.70 \times 7.93 = 132.5$$

and,

$$M_c = +27.10 \times 7.37 = 199.7$$

At the fifth-floor joints the average value of $E\,\theta$ was equal to 4 960. Table 7 begins with no wind moment in the beams and with column moments obtained from approximate lateral movement as suggested by the author. These final results, only, are plotted as Curves 12, 13, and 14, in Fig. 14.

Method 4.—To check the other methods an analysis was made by slope deflections. The labor involved makes this method prohibitive for the actual design of structures. It is useful, however, as a criterion to measure the degree of approximation of the other methods.

In this case, the solution of the resulting simultaneous equations required considerable care, as the wide differences in the relative stiffnesses caused considerable variation in the coefficients, requiring nine significant figures for a satisfactory degree of precision.

The results at Points *a, b,* and *c,* are plotted as constants across all the cycles of Fig. 14 (see Curves 15, 16, and 17.) For comparison with Method 3, the average value of $E\,\theta$ for the fifth-floor joints was 4 920.

Conclusions.—For practical purposes with a bent of three spans of this type, the ordinary assumptions of Fleming's methods give reasonably satisfactory results. They afford the only method for making a preliminary design to use as a basis for more nearly exact elastic computations.

The method of starting with no wind moment in the girders and gradually transferring from the columns into the girders will bring convergence fairly

rapidly in simple, symmetrical bents in which the relative stiffnesses of the different members do not vary too much.

When the members vary widely in their relative stiffnesses, the method of starting with no moment in the girders is too slow and tedious a process. In this particular case, twenty-three cycles of operations failed to come very close to the desired results.

By carrying through a few cycles and then proportioning the final values on the basis of the calculated moment in the columns to the true story moment, a number of cycles is saved. If the proportioning is done three or four times using successive moment values after each cycle, a trend is established which indicates the final value more accurately.

More rapid convergence is obtained by starting with moments as nearly as possible like the final values in the girders. The convergence is rapid, but appears mathematically unsound because there is a marked tendency to converge on the assumed values. If these happen to be nearly correct (and with a little care in selecting them they should be), the results look good, but actually the method does not maintain the right angles between the beams and columns unchanged.

The author's method of approximating the actual story deflections from the relative stiffnesses of the beams and columns combined, and using this deflection to approximate the column moments gives a much more rapid convergence than the original method of moment distribution. The results seem to be satisfactory. The author stated that some variation in relative stiffness was permissible. In this particular case, the stiffest column is more than 2 700% of the spandrel beam—a wide variation, and yet not more than can be anticipated in concrete structures.

Too many practical considerations are involved to make a high degree of precision necessary or desirable. A method that gives the necessary data for selecting members, rather quickly and sufficiently close for practical purposes, is what is desired.

It appears that one of Fleming's methods is quite close. If a check or more precise determinations are desired, the author's suggested method is by far the simplest. Although the tabulated computations appear rather simple, it should be remembered that they represent only three points, and the detailed computations involve several thousand sets. Approximately 75% of the time of Method 1 was saved in Method 4.

While it is unsound to judge from one or two isolated cases, the results as obtained from this one study are very favorable.

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852

DISCUSSIONS

METEOROLOGICAL DATA
PROGRESS REPORT OF SPECIAL COMMITTEE
Discussion

By MESSRS. C. S. JARVIS, C. F. MARVIN, AND IVAN E. HOUK

C. S. JARVIS,[16] M. AM. SOC. C. E. (by letter).[16a]—This report shows unmistakable evidence of earnest efforts directed not only toward improving the service that now procures the data, but also toward suggesting means for the practical use of such information after it has been assembled and published. In the writer's opinion, the latter needs to be stressed, along with some of the commendable features of the United States Weather Bureau methods and the results that are achieved. A casual reading of the "Summary, Including Recommendations" leaves the impression that a list of deficiencies of so broad a scope, if fully warranted, may be counterbalanced by an equally formidable array of constructive suggestions for better use of the voluminous data now available.

A notable service is being maintained at minimum cost throughout the rural and remote sections of the United States by co-operative observers. Among them there are real scientists, according to the personal knowledge of the writer, veterans in the service whose compensation ranges from nothing to only a few dollars per month, including voluntary observers whose personal interest in scientific and natural phenomena alone accounts for their regular and fairly accurate reports. Perhaps undue stress has been placed on the relatively low compensation received by various members of the lower professional and sub-professional grades, in view of the widespread activities of co-operative observers.

Item No. 11, under "Summary, Including Recommendations", apparently condemning the practice of placing the weather-recording instruments on the roofs of buildings, for example, inclines toward excluding such activities from metropolitan areas; or, perhaps, the alternative is to select an open space in a public park for such instruments. Probably the records of co-operative

NOTE.—This Progress Report of the Special Committee on Meteorological Data was presented at the Annual Meeting, New York, N. Y., January 18, 1933, and published in January, 1933 *Proceedings*. This discussion is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

[16] Prin. Hydr. Engr., Engr. Dept. at Large, U. S. War Dept., Washington, D. C.

[16a] Received by the Secretary February 14, 1933.

observers who might be engaged at several locations in the suburbs, with kiosk stations near the ground, would afford the requisite data with which to correct the errors recorded on tops of buildings due to heat radiation, wind, and other disturbances there encountered. It would amount to irreparable loss to abandon the primary stations established on high roofs, even though the higher wind velocities account for a deflection of precipitation, and a reduction in the amounts recorded. Higher than normal wind movements may occur at the street level, as any one will observe at the corners of the U. S. Custom House facing the Battery, in New York City. These velocities are doubtless nearly comparable with those recorded on roofs of moderate elevation.

DR. C. F. MARVIN[17] (by letter).[17a]—Throughout its history the Weather Bureau has accumulated an enormous mass of statistical meteorological data dealing especially with storms, and of climatic data, including precipitation, flood information, etc. These data have been published as fully and completely as possible and in a systematic and homogeneous form, often in co-operation with other national meteorological services of the world, to meet a wide and diversified use on the part of the public and many organizations, including engineers. The activities of the Weather Bureau are outlined very clearly and definitely by specific legislation under which it becomes a Government organization for a distinct public service in the field of meteorology for the benefit of agriculture, commerce, and navigation.

The writer has been personally connected with the work for nearly fifty years, and as Chief of the Bureau has been responsible for the administrative direction of its work for the past twenty years. His views, therefore, are based on a very wide and intimate acquaintance with the entire structure and historical background of its development.

Many errors of fact appear in the report of the Committee, accompanied by evidences of misunderstanding of the operating structure of the Weather Bureau, and a seeming lack of knowledge of the organic laws which impose on it specific duties for the benefit of commerce, agriculture, and navigation. Readers of the report who are not correctly informed as to the organization of the Weather Bureau, its functions, its personnel, its operating methods, and the service it renders, may have been misled and materially misinformed. Accordingly, it seems pertinent to endeavor to correct such misimpressions. A complete answer will not be attempted, but comments will be confined mainly to the numbered paragraphs of indictment as they appear in the report.

Conclusion 1.—Exception is not taken to this "Conclusion," but it would be well to add that engineering forms only a fractional part of the professional, governmental, and private interests which depend on the Weather Bureau for meteorological data.

Conclusion 2.—These allegations which appear to be based on some 200 answers to a questionnaire sent out by the Committee, are vague and too

[17] Chf. of Weather Bureau, U. S. Dept. of Agriculture, Washington, D. C.

[17a] Received by the Secretary March 11, 1933.

indefinite to attempt explicit discussion. They may express the opinion of the Committee which, with the exception of one deceased member from the East, consists almost exclusively of residents of the Pacific Coast. The allegations may also represent the views of some others than the Committee, but the writer is loath to believe that the Engineering Profession as a body subscribes to such opinions. This belief is based on the fact that the Weather Bureau has served engineers for many years and in many ways. Its files contain some criticisms, but many expressions of appreciation. The Weather Bureau does not circularize engineers or any other class of beneficiaries of its service for the purpose of securing commendations or criticisms.

Conclusion 3.—The Bureau does not boast of sensational research in meteorology. By its membership in the International Meteorological Organization and its personal contact with meteorological leaders in all nations, it is fully abreast of the scientific progress of meteorology throughout the world. As a matter of fact, the Weather Bureau is regarded, it is believed, as a leader in the applications of meteorological science to the welfare of man. Moreover, its experts, by personal study and reading, are in the forefront of acquaintance and familiarity with the dynamic and theoretical developments of atmospheric physics, including also the analysis and discussion of meteorological statistics in their relation to agriculture, precipitation, and long-range forecasting. The pages of the *Monthly Weather Review* particularly contain numerous publications by personnel of the Bureau and other specialists of authority dealing with the theoretical and scientific developments and application of meteorology.

In Part VI the Committee undertakes to discuss the influence of elevation on the erroneous catch of rain gauges, including the influence of elevation on wind, temperature, etc. Inferentially, it is assumed that the Weather Bureau is criticized because it has not itself put out such investigations and discussions as these of the Committee and given formulas by which its records of precipitation, wind, temperature, etc., could be corrected on account of what to the Committee seems a grievous fault of Weather Bureau records, especially those in its principal city stations which are collected from instruments at a considerable elevation above ground. As a matter of fact, there are only about 250 stations that have instruments in elevated locations. Moreover, the question of faulty records is a subject which always receives careful attention, and effort is made to secure the best possible exposures in all cases. Space prevents attempting an adequate discussion of this question.

By properly cross-checking city records with records from about 4 000 co-operative observers, the Bureau is convinced that its original records are decidedly more accurate than could possibly result from any such system of corrections as the Committee seems to satisfy itself should be worked out. Accept, for example, the Committee's parabolic law indicated in Fig. 7 for the correction of precipitation, or Fig. 8 for the reduction of wind velocities for elevations above ground. Observing the wide scatter of the original observational values over the smooth parabolic curve one feels convinced that the curve is simply a crude, empirical approximation to the truth. No physical law is represented. To meteorologists acquainted with the real

facts, it is ridiculous to attempt to vitiate original records by applying so-called corrections for elevation of instruments above ground, such as exemplified by these researches of the Committee. As a matter of fact, each station and location must be considered as a law unto itself. Time and again, for example, comparative observations have been made of the rainfall collected on the roof of the Weather Bureau, at Washington (42 ft above ground), with a catch on the ground. The real accuracy of these observations as originally taken is fully proved and to apply any correction to the roof gauge would only destroy and nullify its inherent accuracy. In fact, the Weather Bureau prides itself that it has resisted throughout its history the practice of applying imaginary corrections to observational data. Its fundamental responsibility to the public is to publish the actual observed facts as far as humanly possible. The expert or the specialist of the future is free to mutilate or improve these original facts according to his ideas of what corrections should or should not be applied.

Mention of a striking exception to this practice, will serve to show the sound research efforts of the Weather Bureau. From the beginning of its history the Bureau measured wind velocity with a certain standard type of 4-cup anemometer. Numerous efforts were made to convert the indicated velocities of these instruments into true wind movement, as it was well known that the instrument indicated too high a velocity in high winds. A correction formula and tables giving corrected velocities were published about 1890. This research, however, was based on calibration tests at wind velocities of less than 50 miles per hour. Accordingly, no corrections were applied to original records because of the uncertainty at high velocities. With the recent advent of powerful wind tunnels and high artificial velocities used in aerodynamic studies it has been possible to carry these verification tests to the highest velocities (a maximum of 140 miles per hour). The results of this research have now fully justified the Weather Bureau for the first time not only in stating confidently the law of performance of the 4-cup anemometer, but also in correcting observed wind records to true wind records at the instrument as exposed. This practice began January 1, 1932. Engineers now have true wind velocities within a small margin of probable error.

Conclusion 4.—In this criticism the members of the Committee display a serious lack of knowledge of the necessities and requirements of a meteorological service and of certain prerequisites in the preparation of meteorological data. These, to be of value, must be homogeneous and must be presented uniformly in a standard method arrived at, as a matter of fact, very largely by organized co-operation on the part of the meteorologists of all nations. Who would tolerate the captious alteration of uniform routine and mechanical methods of presenting meteorological statistics, the value of which increase enormously with the homogeneity, uniformity, and great length of the record of observations? The Bureau claims that it deserves commendation; that for many years its records have been presented in a homogeneous and uniform manner. Criticism would be justified if every imaginary new need and so-called new condition as they arise led to changes of compilation and preparation.

It is incumbent on the Weather Bureau to present its data in a way that will meet the needs of all the people. Manifestly, it is not possible to arrange data for publication in multitudinous forms to suit the convenience of everybody. Such procedure would result in an enormous and prohibitive cost. The present arrangement is the development of more than sixty years. It is confidently believed that the Bureau is following a proper course when it collects and disseminates its data in a form which meets general requirements and, at the same time, permits those desiring special compilations to make re-arrangements to suit their respective needs.

It is clear that the Committee in Item (*c*) of this criticism concerning short-time forecasting takes no cognizance of the organic laws which impose specific duties on the Weather Bureau for the benefit of agriculture, commerce, and navigation. Forecasting the weather comprises a large and essential part of those duties. Weather forecasting was the incentive which actuated the establishment by law of a national meteorological service in the United States. The issue of forecasts and warnings, including information on current weather, constitutes functions of service which are of primary importance. Nevertheless, the Bureau is also specifically enjoined by law to "take such meteorological observations as may be necessary to establish and record the climatic conditions of the United States." Accordingly, the apparent implication by this passage in the report of the Committee that climatological data and records have been neglected, has no foundation in fact. Forecasts are predicated on telegraphed reports. Forecast service would be impossible without reports by telegraph, but these data become a component part of the climatological record made at the same time. Reports are telegraphed from about 250 stations in the United States. On the other hand, the climatological records with which engineers are concerned are based on observations made at about 4 500 stations. The forecasting work and climatological work, while interrelated in some ways, are distinct projects, one being as thoroughly organized and operated as the other.

Conclusion 5.—This "Conclusion" deals with an academic and debatable question. Nothing would be gained by discussing it in detail. The activities of the Weather Bureau, throughout its history, have been conducted with the greatest possible economy and efficiency. Certainly, within the present century, its rapid advance in service to all agencies has been devoid of waste and extravagance in every particular. Furthermore, no undue restraints or restrictions have been placed upon any leading member of its organization. The tendency of modern times is to resist subdivision of duties and responsibilities and to avoid opportunities for wasteful duplication and extravagance. It is insisted that the present so-called centralized organization of the Weather Bureau satisfies the present standard of economy and efficiency in management. Meteorology is one of the few activities of the Government that is controlled by one central organization with practically no duplication on the part of other agencies. Taken in its vague implications the criticism of the Committee means only loose organization with wide freedom of independent action and needless duplication of effort.

Here, again, there seems to be evidence of serious lack of information on the part of members of the Committee of the real organization, structure, and operation of the Weather Bureau. In this connection mention must be made of the gross inaccuracy in the tabulation in Part V under "Organization," purporting to state what the present activities of the Weather Bureau cover.

The Committee has drawn this information from a private publication[*] by D. Appleton and Company, dated 1922. The activities listed are really marginal headlines of paragraphs in a chapter of the original private monograph discussing in general terms the activities of the Weather Bureau as of the date, 1922. As the context of the monograph shows, a number of these alleged activities were certain incidental items of work and the latter part of the list covers allusions to topics discussed in certain publications that happened to be prominent in 1922. Moreover, some of the items have been entirely taken over by other agencies since 1922. For example, "Reporting effects on weather upon highways" was inaugurated and carried on as a World War activity by the Bureau, but subsequently left to be handled by automobile and other private interests; "Studies in seismology" and "Studies in volcanology" were voluntarily relinquished by the Weather Bureau, through the co-operation of the Director of the Budget, as soon as it became possible for other agencies of the Government to carry on the work. The same thing is true of "Maintaining and operating telegraphic lines." It is unfortunate the Committee did not inform itself in a more up-to-date manner with regard to present organization of Bureau activities.

In this same connection a brief comment is made on the specific recommendation of the Committee No. 7 (following the "Conclusions"), that the Bureau be re-organized so as to allow for a grouping of the present sixteen administrative branches into from three to five divisions, and also to allow more authority, responsibility, and initiative to be exercised by local officials.

Just what are included in the so-called sixteen administrative branches mentioned by the Committee is not clear and in this recommendation it is difficult to escape the impression that the Bureau is criticized, on the one hand, for too much centralization of authority in Washington and, on the other hand, for too great a subdivision of administrative supervision. Nearly sixty years of experience, including exactions of appropriation legislation, budgeting, and accounting for expenditures, etc., have led to the natural and rational grouping of responsibilities of the Weather Bureau in certain logical fields. There are, for example, six major divisions concerned with the service and technical work of the Bureau, as follows:

Forecasts: General and special forecasts and warnings with Forecast Centers at Washington, D. C., Chicago, Ill., Denver, Colo., San Francisco, Calif., and New Orleans, La.

Climate and Crop Weather: Comprises more than 4 000 climatological field stations locally supervised at 42 Section Centers, generally one for each State. These same local Sections supervise the weekly crop weather reporting.

River and Flood: River and flood forecasts and warnings; controls more than 700 river gauge and rainfall stations mainly decentralized among 63 field districts; compiles river-stage data.

Marine Meteorology: Comprises compiling and charting data contained in logs and reports from more than 1 000 ships plying all the oceans of the globe. Furnishes meteorological data for Pilot Charts published by the Hydrographic Office.

Aerology: Provides, in many cases, 24-hour service to aviation over approximately 25 000 miles of official civil airways. Technical digest and discussion of data as accumulated.

Instrument: Devoted to selecting, testing, exposing, and utilizing instruments maintained at stations.

Numerous collateral projects, more or less distinct in themselves, come within the purview of the aforementioned six divisions.

It is difficult to conceive of a more condensed or rational subdivision of administration of Weather Bureau activities than the foregoing. Of course, there are general administrative offices common to all large organizations (Chief of Bureau, Assistant Chief, and Chief Clerk), and, in addition, a number of minor sections controlling accounts and disbursements, maintenance of equipment for field printing, procurement of supplies, etc., that are essential in the prompt and efficient dispatch of work, but should not be listed as distinctly administrative responsibilities.

The Committee offers no explanation whatever for its recommendation; nor does it justify it by any citation of efficiency or economy. Apparently, such recommendation is nothing more than an opinion, while the present organization of the Weather Bureau is the result of many years of experience and adaptation of the activities of the Bureau and the administration of specific laws and appropriations. Expansion of this organization would probably lead to greater expenditures for salaries and personnel, offices, fixtures, etc., while any material concentration would be attended with little or no economy or efficiency. Finally, it is not at all obvious or clear how the contraction would confer more authority, responsibility, and initiative on the part of local officials.

Conclusion 6.—The activities of the Weather Bureau can not be satisfactorily compared with those of any other governmental organization. Its functions involve a form of service provided by no other agency, governmental or private, and its personnel necessities are peculiar to itself. Technical investigations and research are essential, of course, but it is mainly a public service organization. Personnel of the Bureau is adjusted in a manner to best meet those needs.

Inference is made that employees of the Weather Bureau in general should have college degrees and should be qualified for research in pure and applied sciences. Such organization would not be advocated by any one familiar with the operations of an enormous network of meteorological field stations. It is practicable to prosecute original scientific work at only a few of them. This is done. Operations at a very large majority of field stations consist of taking

observations, recording and telegraphing them, printing weather maps and bulletins, and disseminating weather information, forecasts, and warnings to areas served by the respective stations. Therefore, most of the work, although it requires men of good education, is not of a character to justify that the whole or a considerable part of the personnel be specifically engaged in research work for which facilities and possibilities are greatly limited.

Exclusive employment of many technically educated and trained men would materially increase salary costs, and real efficiency would be sacrificed. Men educated for and qualified to do research work would not be content to perform the large amount of relatively routine work that is required. However, the Weather Bureau does have capable and technically trained research workers who, for the most part, are and necessarily must be centralized at Washington and a few major field stations.

Conclusion 7.—The writer has no disposition to debate the statement that "appropriations for the support of the general activities of the Bureau (not including aerological work) have maintained a reasonable scale of increase in the past"; nor would he be inclined to do so if "aerological work" had been included therein.

Conclusion 8.—In this statement which implies that "the salary ranges * * * appear to be quite fair," but that "the under-classification of employees is apparent" there seems to be a bit of obscurity and contradiction and the members of the Committee do not appear to understand that the entire classification of employees in the Government, in Washington especially, and, as far as practicable, in the field also, is specifically defined categorically by the Classification Act. The whole question of classification of personnel in Government departments is complicated and involved, but settlement of such classifications does not rest with the head of a Bureau. He can only strive to secure for his subordinates a fair compensation according to ability, industry, and work performed, and earnest effort in that direction is the policy and practice in the Weather Bureau. Undoubtedly, there are instances in which employees are not properly compensated, but such is the case in many other organizations, governmental and private. It is a super-human task to secure satisfaction or even justice in all cases.

Exception must be taken to inferences that there is a low spirit of morale in the Weather Bureau. This is not the case. It is confidently believed that the *esprit de corps* in the Weather Bureau organization, especially among the older and leading members, is not surpassed by that of any branch of the Government. A great preponderance of its employees would resent an imputation that the morale of the Bureau is low.

Conclusion 9.—Implication that there is a conscious negligence or indifference in the selection and distribution of stations of the Bureau is unwarranted. Even in the mere fragment of 200 principal stations located in the principal cities of the country for the purpose of accomplishing the specific objects of legislation, the Bureau itself has often been compelled to locate stations closer together or more scattered than it would otherwise choose. On the other hand, the method of selecting more than 4 000 co-operative observers, which, of course, has grown up largely with the spread of population

of the country from the east westward, choice of stations, and locations, is limited and no human power can exercise an ideal choice of distribution. Moreover, it is denied that the distribution either fails to represent fairly, or that it misrepresents, the essential climatological characteristics of the entire country. In this connection the chart contained in Part II, Atlas of American Agriculture, as compiled by the Weather Bureau, should be consulted. The chart shows the location of all the observation stations in the United States in 1922 and is still representative of the station distribution. It is assumed that engineers are acquainted with this publication.

It is recognized that in some areas records are obtained from more places than others, but the reasons for this should be obvious. There are large areas, especially in the mountainous and arid sections, in which more stations would be advantageous, but such places are sparsely inhabited or are uninhabited during a considerable part of the year, or they are not habitable at all. Therefore, continuity of records is not possible in some areas. In others, they could not be secured without providing living quarters and special assignment of observers. This would involve a prohibitive cost for salaries, upkeep, and operation. At present, 4 503 stations are maintained, representing as far as possible every topographic feature of the country, ranging from 185 ft below sea level to more than 10 000 ft above sea level.

Conclusion 10.—Practically all instruments used at all Weather Bureau stations, including co-operative stations, are Government-owned, are of the highest class, and are tested before being placed in use. Inspections are made as often as are warranted. Prompt action is taken to remedy any unsatisfactory conditions.

Each Section Director, when the monthly reports are in from co-operative observers in his State, carefully examines the data and checks each station's record against similar reports furnished from neighboring stations to insure as far as humanly possible the accuracy of the data to be published. Questionable records are not used.

A great majority of stations operate on a co-operative basis; that is, the observers do not receive any monetary compensation for their work. This plan is economical and wise. The number of observers is very large. If given pay in excess of a ridiculous pittance, the cost would be prohibitive. It is reasonably certain that if it were possible to pay even a small compensation to each co-operative observer many persons would be eager to secure a position as observer principally for the pay involved. Such observers could not be depended upon to give the work the careful attention now secured from persons who seek the work solely for the interest they have in it. Many cases can be cited in which such volunteers have spent nearly a lifetime in faithfully making these records.

The Bureau does not deny that among more than 4 000 co-operative stations making observations without pay of any kind, some cases of faulty exposures and possible imperfect records do arise. These constitute a fraction of a per cent. of the mass of observational data, and an attack on the integrity of the records on this basis is unwarranted.

Conclusions 11 and 12.—Proper exposure of meteorological instruments is a matter of high importance. It is given careful attention and effort is made to secure the best possible exposure in all cases. It is practically impossible to secure ideal exposures in cities. To obtain permanently uninfluenced sites for instruments in cities, it would be necessary to purchase a large area of land in each case, install the instruments in the center, and permit no interfering changes to take place. Manifestly, this is not practicable, nor is it humanly possible to anticipate long in advance the occurrence of conditions which require change in location. Occasional changes in locations can not be avoided. What was considered a good exposure at the time of installation may become a decidedly unsatisfactory one because of growth of the city and other causes beyond control. When changes in location of instruments become necessary comparison is made between observations taken at the old and the new site whenever it can be done and for as long a period as possible.

It is well known that to secure correct records of precipitation gauges must be placed where they are least affected by prevailing air motions (winds), since currents and eddies around and about the mouth of the gauge cause errors of catch, usually deficiencies. Various devices are used to minimize the effect of these conditions.

Comment has already been made of that part of the report of the Committee which is devoted to formulating a law of corrections for the exposure of rain gauges, wind instruments, and thermometers at various elevations above ground. The effort is really amateurish as it fails to recognize the *de facto* conditions of each instrumental exposure and presupposes that the only influence vitiating the record is an elevation of 50 ft to a few hundred feet above ground. Of the 4 503 stations at which observations are made, the instrumental equipment at more than 95% of them is placed on or near the ground. These require no correction. A small number of remaining stations are located in cities where serving local needs is the dominant factor. The avoidable errors in these records are as small as practicable. Records from ground exposure at more than 4 000 stations afford an accurate check on any serious local inaccuracies.

Even if it were possible to develop correction formulas for the variations in exposure of rain gauges and anemometers during a period of more than sixty years and to publish them in connection with the data, it would create a hopeless state of confusion and would make the data difficult of use to a majority of those who utilize them. A serious investigator can call upon the Bureau for any details he may desire, and they will be furnished. It is better that an investigator be put to the trouble of writing a single letter than to use the people's money in publishing voluminous records of changes and corrections that would be used by very few. Monthly and annual publications of data, and also the summarized tables in *Bulletin W*, show the elevation of each station.

Meteorology, including climatology, is a distinct science, allied to many, but having its own specialized field. Activities of the Bureau are adminis-

tered by officials of long experience, who are well informed as to the requirements of the many interests that are served. Criticism and constructive suggestions are welcomed from any one regarding the lines of work with which he is directly concerned, but recommendations for a complete re-organization of a large Government Bureau, in order to secure a favored form of service to the disadvantage of a large majority of other beneficiaries, do not justify serious consideration.

The Committee specified qualifications which should be followed in selecting a Chief of the Weather Bureau. Some desirable attributes are described, but this is an elastic question capable of numerous variations, according to individual viewpoints. Slight consideration seems to have been given to the eligibility of meteorologists, or of those who have had training in the science of meteorology and experience in its practical application to agriculture, commerce, and navigation. It is likely that when the question of making appointment to the office arises, the appointing authority will wish to make his own determinations of fitness and it does not seem ethical or proper to inject discussions of this kind in a report which ostensibly had for its purpose improvement in a public service.

Engineers are held in high respect by officials of the Weather Bureau, who desire and endeavor to serve them freely and efficiently. The writer is loath to believe that widespread feeling of dissatisfaction exists in the Engineering Profession regarding the work of the Weather Bureau, or that engineers in general would expect this large organization, having both a National and international scope, to warp its entire structure because of reasons given in the report.

Ivan E. Houk,[18] M. Am. Soc. C. E. (by letter).[18a]—The Committee's comprehensive progress report shows that the members, either personally or through their engineering assistants, have made a detailed study of the work of the Federal Weather Bureau. Undoubtedly, some of its conclusions and recommendations are justified. However, the writer feels that the report is not, in all cases, entirely fair to Government meteorologists.

The writer does not believe that Item 2 of the "Summary" is justified. The Engineering Profession, as a whole, probably believes that some improvement could be made in the location and distribution of stations, the quality of records, and manner of publication; but it is very doubtful whether this feeling can be properly expressed as one of "widespread dissatisfaction." It is quite likely, for example, that the sixty-three members of the Society who did not reply to the questionnaire do not have a definite feeling of dissatisfaction with regard to Federal meteorological work. If they had had such a feeling, they undoubtedly would have replied. Such is human nature. People usually do not take the trouble to commend public servants when such commendation is warranted; but they do take the trouble to criticize whenever an opportunity presents itself. If the weather forecast goes wrong, every one noticing it mentions the fact; but if the forecast is verified (as it is

[18] Senior Engr., U. S. Bureau of Reclamation, Denver, Colo.

[18a] Received by the Secretary March 15, 1933.

approximately 90% of the time), the people take it as a matter of course and no one says anything about it.

Fig. 6, showing the geographical distribution of United States Weather Bureau stations, does not bear out Item 9 of the Committee's conclusions. Fig. 6 shows a remarkably satisfactory distribution of stations. Considering all the various uses of meteorological data, including those of an agricultural, economic, and industrial nature, as well as those connected with engineering, it is quite logical that there should be a preponderance of stations in the more thickly settled parts of the country. The writer is surprised that the geographical distribution throughout the country is as even as it is. There might reasonably have been a considerably greater preponderance of stations in the Northeastern States.

Additional stations in the higher mountains are probably desirable from the standpoint of determining relations between altitude and weather; but the writer feels that the geographical distribution, as a whole, is sufficient to furnish a fairly satisfactory presentation of climatological characteristics.

Since about 1913 the writer has made many detailed studies of practically all kinds of meteorological data, including, particularly, the records of rainfall, snowfall, run-off, temperature, wind movements, relative humidity, and evaporation. There have been times when he has regretted that more complete weather data were not published. There have been times when he has found it necessary to secure unpublished information from regular Weather Bureau stations, either by correspondence or through personal interviews. There have been times when he has needed additional data not available in the Weather Bureau offices. However, on the whole, he has been very well satisfied with the quality of the weather records and the manner in which the data have been published, particularly during the more recent years.

The Committee seems to be unduly worried because of the fact that the instrumental equipment at most of the regular Weather Bureau stations is located on the roofs, or above the roofs, of high office buildings. The report cites a number of instances in which meteorological data observed at the ground level, at locations relatively close to the elevated stations, differed from the data observed at the higher levels, particularly as regards records of wind, precipitation, and temperature. There is nothing new about this. The fact that such variations exist has long been known by engineers who have made detailed studies of climatological data as well as by meteorologists engaged in weather measuring work. What of it? The existence of such variations does not mean that satisfactory climatological data are not being obtained. It simply means that it is desirable, whenever possible, to maintain kiosks in open municipal park areas, as the Weather Bureau is now doing in several instances, so that the meteorological relations between the two locations can be established.

The writer feels that, in many cases and for many purposes, it may be more desirable to have the climatological record maintained at the top of a high office building than it is to have it maintained at the ground level. He certainly would not favor the maintenance of a meteorological station at the

center of a busy street intersection, surrounded by high office buildings. Neither would he favor the maintenance of a meteorological station in a small park area surrounded by relatively high building developments, unless a duplicate station could be maintained on the roof of one of the higher buildings.

As a matter of fact, meteorological conditions in a large municipality usually vary appreciably in different parts of the city as well as at different heights above the ground. Some districts may experience greater wind movements than the elevated Weather Bureau station. Others may experience lesser wind movements. The same is true of temperature, precipitation, and other weather phenomena. Such meteorological variations are due to differences in overhead air and storm movements and local, ground-level differences in altitude, topography, vegetation, trees, building developments, etc. Since these meteorological variations do exist, and since they are manifestly of an irregular nature and origin, there is no mathematical formula for determining an exact location in a large municipality where the weather station should be established in order best to represent the average climatological conditions. The location must be made on the basis of scientific judgment. Who can be better fitted to exercise such judgment than the meteorologist who has made weather phenomena his life study?

Probably the encouragement of research activities, by co-operation with scientific institutions as well as by the Bureau meteorologists themselves, is one of the most important matters recommended by the Committee. There is no doubt but that additional meteorological research is needed. This is true whether the matter is considered from the standpoint of agricultural, economic, and industrial interests, or from the standpoint of engineering. Such research activities should include comprehensive studies of the meteorological records already collected, as well as detailed studies of the fundamental laws involved in weather phenomena and the possibilities of making long-range weather forecasts. The institution and prosecution of a comprehensive research program is much more important than the establishment of new stations, either within the larger municipalities, or in the more thinly settled parts of the country. It is also much more important than the publication of detailed records of changes made in location of equipment at existing stations, or the institution of an elaborate system of co-operative station inspection.

APPLICATIONS FOR ADMISSION AND FOR TRANSFER

The Constitution provides that the Board of Direction shall elect or reject all applicants for *Admission* or for *Transfer,* and, in order to determine justly the eligibility of each candidate, the Board must depend largely upon the Membership for information.

This list is issued to members in every grade for the purpose of securing all such available information, and every member is urged to scan carefully each monthly list of candidates and to furnish the Board with data in regard to any applicant which may aid in determining his eligibility. It is the *Duty* of all *Members* to the *Profession* to assist the *Board* in this manner.

It is especially urged, in communications concerning applicants, that a *Definite Recommendation as to the Proper Grading in Each Case* be given, inasmuch as the grading must be based upon the opinions of those who know the applicant personally, as well as upon the nature and extent of his professional experience. If facts exist derogatory to the personal character or to the professional reputation of an applicant, they should be promptly communicated to the Board. *Communications Relating to Applicants are considered by the Board as Strictly Confidential.*

The Board of Direction will not consider the applications herein contained from residents of North America until the expiration of thirty (30) days, and from non-residents of North America until the expiration of ninety (90) days from April 15, 1933.

MINIMUM REQUIREMENTS FOR ADMISSION

Grade	General Requirement	Age	Length of Active Practice	Responsible charge of work
Member	Qualified to design as well as to direct important work	35 years	12 years*	5 years of important work
Associate Member	Qualified to direct work	27 years	8 years*	1 year
Junior	Qualified for sub-professional work	20 years†	4 years*	
Affiliate	Qualified by scientific acquirements or practical experience to co-operate with engineers	35 years	12 years*	5 years of important work
Fellow	Contributor to the permanent funds of the Society			

* Graduation from a school of engineering of recognized reputation is equivalent to 4 years of active practice.

† Membership ceases at age of 33 unless transferred to higher grade.

The fact that applicants give the names of certain members as references does not necessarily mean that such members endorse.

FOR ADMISSION

ALLEN, TOM JOHNSON, San Diego, Cal. (Age 46.) Allen & Rowe, Civ. Engrs. Refers to G. Butler, E. A. Ingham, T. H. King, R. W. Randolph, R. R. Rowe, H. N. Savage, C. H. Splitstone.

ATKINS, ROBERT BOYER, Washington, D. C. (Age 21.) Asst. Constr. Engr., Aronberg-Fried Co., Bldrs. Refers to H. C. Bird, E. A. Steece, W. R. Weidman.

BEACHAM, JACK GARLINGTON, Athens, Ga. (Age 27.) City Engr. and Supt. of Water-Works. Refers to J. W. Barnett, H. Beebe, C. R. Hopper, W. D. Hull, T. M. Neibling, S. B. Slack, C. M. Strahan.

BHATT, UPENDRA JIVAHRAM, Bhavnagar, India. (Age 23.) Refers to T. R. Camp, W. M. Fife, K. C. Reynolds, C. M. Spofford, J. B. Wilbur.

BROWN, EDGAR GREGSON, Detroit Mich. (Age 33.) Jun. Engr., U. S. Engr. Office, War Dept. Refers to H. C. Corns, F. B. Duis, C. James, R. E. Mackenzie, W. E. Sanford, E. E. Teeter.

BROWNELL, EDWARD FULLER, La-Crosse, Wis. (Age 33.) Asst. Engr., U. S. Engr. Office, St. Paul, Minn. Refers to O. E. Brownell, A. S. Cutler, A. J. Duvall, G. O. Guesmer, H. M. Hill.

CLARK, EDWARD SHANNON, East Worcester, N. Y. (Age 30.) County Highway Surveyor, Otsego County. Refers to E. R. Cary, L. W. Clark, W. W. Rousseau, W. C. Ruland, H. O. Sharp.

DICKSON, RONALD, Capetown, South Africa. (Age 32.) With Rys. and Harbors Administration, Union of South Africa. Refers to N. Shand, R. J. van Reenen, C. V. von Abo. (Applies in accordance with Sec. 1, Art. I, of the By-Laws.)

FRAAD, HENRY OBER, Great Neck, N. Y. (Age 33.) With Allied Pneumatic Services, Inc., New York City. Refers to D. Y. Geddes, U. S. Grant, 3d, R. R. Nace, J. L. Nagle, C. L. Spaulding.

GARRAN, FRANK WARREN, Hanover, N. H. (Age 38.) Asst. Prof. of Civ. Eng., Thayer School of Civ. Eng., Dartmouth Coll. Refers to J. H. Dingle, T. W. Dix, R. Fletcher, R. R. Marsden, C. H. Sutherland, J. T. Whitney, A. E. Winslow.

HERMAN, MURRAY, New York City. (Age 26.) Eng. Asst., Grade 3, Dept. of Parks, Brooklyn, N. Y. Refers to J. S. Peck, J. C. Rathbun.

HILL, GEORGE MITCHELL, Riverside, Cal. (Age 43.) Gen. Mgr., Parker Machine Works. Refers to D. M. Baker, N. Bostwick, D. A. Doble, F. S. Foote, H. Hyatt, P. J. Lynch, E. M. Scofield.

JOBES, JAMES GIBSON, Vicksburg, Miss. (Age 26.) Asst. Engr., U. S. Waterways Experiment Station. Refers to G. R. Clemens, T. H. Jackson, H. W. King, G. H. Matthes, P. S. Reinecke, H. D. Vogel, C. O. Wisler.

KAMY, HARRY DONALD, New York City. (Age 24.) Refers to J. B. Babcock, 3d, C. B. Breed, J. W. Howard, K. C. Reynolds, C. M. Spofford.

KAVANAGH, THOMAS CHRISTIAN, New York City. (Age 20.) Refers to R. E. Goodwin, F. O. X. McLoughlin, J. C. Rathbun.

KETTLE, KENATH AUSTIN, Charleston, W. Va. (Age 27.) Superv. Engr., South Charleston Constr. Div., Carbide & Carbon Chemicals Corporation. Refers to F. W. Daniels, H. J. Hassler, H. S. Jacoby, C. T. Morris, J. R. Shank, R. E. J. Summers.

KNAP, HANS JØRGEN, Oakland, Cal. (Age 27.) Refers to F. Auryansen, J. W. Stewart.

LESLIE, JAMES BOOTH, Jr., Vicksburg, Miss. (Age 27.) Jun. Engr., U. S. Waterways Experiment Station. Refers to C. A. Baughman, J. A. C. Callan, G. R. Clemens, H. H. Houk, G. N. Mitcham.

MacGREGOR, ROSS EDWARD, Floral Park, N. Y. (Age 36.) Refers to S. M. Ellsworth, S. Gordon, N. C. Holdredge, A. H. Pratt, J. A. Ward.

McCOMB, FRED ROBERT, Wichita, Kans. (Age 22.) Refers to W. K. Hatt, G. E. Lommel, G. P. Springer, R. B. Wiley.

McINNES, JOHN PAXTON, Toledo, Ohio. (Age 28.) Instrumentman, City Engr's Office. Refers to W. S. Dix, C. E. Hatch, L. T. Owen, R. H. Randall, G. N. Schoonmaker, A. H. Smith, R. C. Sweeney, G. D. Whitmore.

MAZZOLA, LOUIS, Brooklyn, N. Y. (Age 33.) Refers to N. D. Brodkin, A. Haring, J. H. Quimby, C. T. Schwarze, T. F. Weiss.

PEGADO, HENRIQUE, Sao Paulo, Brazil. Age 39.) Henrique Pegado & Cia, Ltd. Refers to W. Charnley, C. P. Conrad, J. G. Hollman, A. A. da Motta, J. T. de Oliveira Penteado, A. Y. Sundstrom, W. L. Zeigler.

PIPER, CLAYTON LEO ROY, Toledo, Ohio. (Age 32.) Engr. for Port Comm., City of Toledo. Refers to C. M. Fyler, C. E. Hatch, L. T. Owen, R. H. Randall, G. N. Schoonmaker, A. H. Smith, G. A. Taylor, L. K. Whitcomb, Jr., G. D. Whitmore.

PLATT, HOWARD CHARLES, Tenafly, N. J. (Age 33.) Computer, U. S. Coast and Geodetic Survey, Dept. of Commerce, New York City. Refers to A. B. Cohen, T. Human, Jr., E. P. Leclercq, J. J. Loeser, H. H. Pitcairn, E. J. Squire.

PONSY, KARL WILLIAM, Youngstown, Ohio. (Age 33.) Asst. Chf. Draftsman, Truscon Steel Co. Refers to W. T. E. Barber, H. Barnes, W. Bertwell, C. J. Kennedy, C. Klaesius.

QUENEAU, ROLAND BLAISDELL, New Rochell, N. Y. (Age 30.) Field Engr., The Pitometer Co., New York City. Refers to H. E. Beckwith, C. R. Bird, E. D. Case, E. S. Cole, I. E. Matthews, E. K. Wilson.

RUMSEY, RICHARD MORDEN, Niagara Falls, N. Y. (Age 39.) County Supt. of Highways, Niagara County, N. Y. Refers to J. W. Fortenbaugh, C. L. Oelkers, W. W. C. Perkins, J. H. Sturdevant, G. F. Unger.

SYKES, ROY JAMES, Wichita, Kans. (Age 22.) Refers to W. K. Hatt, S. C. Hollister, W. E. Howland, G. E. Lommel, G. P. Springer, R. B. Wiley.

URBINO, JACINTO, San Juan, Puerto Rico. (Age 36.) Engr., Water-works Bureau City of San Juan. Refers to E. Baez Rodriguez, J. M. Canals, M. Font, S. Quinones, R. Ramirez, E. Totti y Torres.

von WEYMARN, PETER, New York City. (Age 52.) Refers to H. J. M. Baker, B. A. Bakhmeteff, J. S. Butler, A. E. Crane, B. Dibble, C. H. Paul, Z. H. Sikes.

WILSON, CHARLES RUDD, Jr., Portland, Ore. (Age 36.) Asst. Mgr., Creosoting Plant, Charles R. McCormick Lumber Co. Refers to G. W. Buck, F. T. Crowe, F. T. Fowler, P. Hart, B. M. Howard, O. Laurgaard, R. E. Mieth, H. E. Plummer, J. H. Polhemus, D. H. Rowe, F. C. Schubert, O. E. Stanley.

FOR TRANSFER

FROM THE GRADE OF ASSOCIATE MEMBER

GREEN, HOWARD WHIPPLE, Assoc. M., Cleveland, Ohio. (Elected Junior June 18, 1918; Assoc. M., Oct. 15, 1923.) (Age 39.) Secy., Cleveland Health Council and Director of Statistics and Research. Refers to G. C. Bunker, G. B. Gascoigne, R. Hoffmann, R. F. MacDowell, E. F. Sinz, G. B. Sowers, W. J. Watson.

HOLMES, HAROLD RISTINE, Assoc. M., Milwaukee, Wis. (Elected March 16, 1925.) (Age 48.) Chf. Field Engr., Bureau of Sewers. Refers to J. L. Ferebee, T. C. Hatton, G. M. Hinkamp, R. R. Lundahl, C. U. Smith, D. W. Townsend.

HOOPER, ELMER GUY, Assoc. M., New York City. (Elected Nov. 21, 1921.) (Age 50.) Prof. of Hydraulics, New York Univ.; Cons. Engr. Refers to H. E. Breed, J. F. Sanborn, T. Saville, C. T. Schwarze, C. H. Snow, B. H. Wait.

MILLER, JOHN ANDERSON, Assoc. M., New York City. (Elected Junior June 19, 1922; Assoc. M., Feb. 25, 1924.) (Age 37.) Editor, *Transit Journal.* Refers to J. A. Beeler, W. T. Chevalier, F. W. Doolittle, J. H. Hanna, C. R. Harte, E. M. T. Ryder, F. E. Schmitt, O. Singstad.

MOORE, LEWIS BLAFFER, Assoc. M., Balboa Heights, Canal Zone. (Elected Aug. 27, 1928.) (Age 35.) Asst. Office Engr., Sec. of Office Engr., Municipal Div., Panama Canal. Refers to G. C. Bunker, H. Burgess, J. G. Claybourn, W. B. Godfrey, R. C. Jones, E. S. Randolph, J. L. Schley.

FROM THE GRADE OF JUNIOR

GREENEFEGE, SERGE JOSEPH, Jun., Flushing, N. Y. (Elected Nov. 10, 1930.) (Age 29.) Asst. Engr., New York & Queens Elec. Light & Power Co. Refers to R. P. Gustin, J. I. L. Hogan, H. Holbrook, B. Kipp, C. C. Kohlheyer, W. A. Mellny, W. H. Walker.

JACOBS, HARRY VICTOR, Jun., Washington, D. C. (Elected Dec. 22, 1930.) (Age 32.) Res. Engr. and member of firm, W. N. Brown, Inc. Refers to F. J. Biele, W. N. Brown, J. B. Gordon, T. E. Ringwood, M T Singleton, S. L. Thomsen.

NELSON, SAMUEL BALDWIN SMITH, Jun., Los Angeles, Cal. (Elected April 18, 1927.) (Age 30.) Asst. Field Engr., Dept. of Water and Power, City of Los Angeles. Refers to C. E. Angilly, Jr., E. A. Bayley, W. H. Cates, F. W. Hough, N. M. Imbertson, H. L. Jacques, D. A. Lane, R. R. Proctor, W. W. Wyckoff.

The Board of Direction will consider the applications in this list not less than thirty days after the date of issue.